Technical Report **175**

# THE EFFECTS OF LONG-TIME LOADS ON PRESTRESSED CONCRETE HOLLOW-BOX BEAMS

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THE EFFECTS OF LONG-TIME LOADS ON PRESTRESSED CONCRETE HOLLOW-BOX BEAMS

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by

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## OBJECT OF TASK

To determine the effects of long-time tests on prestressed concrete hollow-box beams under loads of various magnitudes.

## ABSTRACT

Prestressed concrete beams suffer from certain time-dependent changes. They lose part of their initial prestress, their deflection increases, and the concrete shrinks and creeps. Information was needed on these long-time effects so that they could be provided for in the design.

To obtain this information on prestressed concrete hollow-box beams, eight such beams were loaded for 4-1/2 years and changes in deflections, concrete strains, and prestressing forces were recorded. The beams were 42 feet long, 33 inches deep, and 18 inches wide. They were simply supported on a 40-foot span and uniformly loaded with concrete weights. Four different loading conditions were used on the eight beams.

The loaded beams had an additional time-dependent deflection during the 28 days following loading approximately equal to the immediate elastic deflection at the time of loading. The beams with no live load continued to deflect upward. Measurements showed that the time-dependent strains in the concrete were greater than the immediate strains due to normal working stresses. The prestressing force data indicates that the unloaded beams experience a greater reduction of prestress than the loaded beams, that the first-year prestressing losses can be as high as 13 percent, and that in 4-1/2 years they can be as high as 22 percent.

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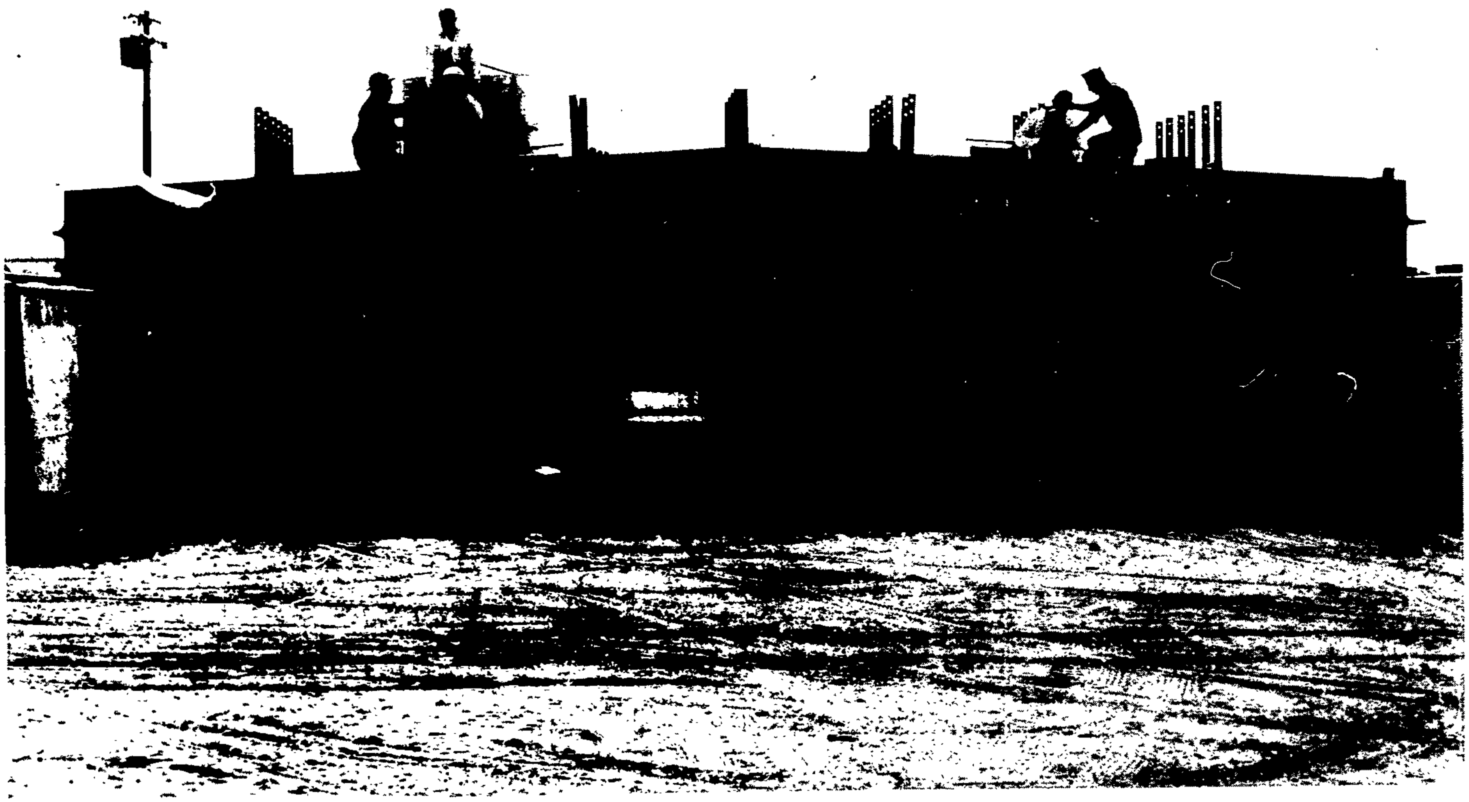


Figure 1. Hollow-box beam being loaded.

## INTRODUCTION

Under long-time loads prestressed concrete structures undergo a number of changes in the properties of the concrete, the dimensions and deflections of the structure, and the prestressing forces that were initially applied. A number of the changes are detrimental and must be allowed for in the design of the structure. It was the objective of this investigation to determine the long-term effects of various loads on prestressed concrete hollow-box beams, and thereby to provide the future designers of such beams with some of the information necessary to execute a more efficient design.

One of the most important changes to occur in prestressed concrete is the loss of prestressing force which develops over a period of time. The basic principles of prestressing were first applied to concrete in about 1886. These initial applications were unsuccessful, however, because of the loss of the prestressing force. This loss is a function of the concrete shrinkage and the creep in the concrete and the steel prestressing units. If the stress in the tendons is low, it may be completely lost.

Prestressed concrete first became practical with the use of high-strength steel wires in France in 1928. These wires could be highly stressed, and retained most of the prestressing force even after all losses had occurred. It was not, however, until 1939 that satisfactory end anchorages were developed. In 1945, after World War II, prestressed concrete rapidly developed in Europe. In 1949 the first prestressed concrete structure was started in this country. By 1951 fewer than ten such structures had been constructed in the United States.[1]

In 1953 this Laboratory started long-time tests on 22 full-sized prestressed concrete I-beams, and in 1955 the investigation of hollow-box beams was initiated. Figure 1 shows the manner in which the loads were applied and the general test arrangement.

## EXPERIMENTAL PROGRAM

### Description of Beams Tested

The beams were all 42 feet long, 18 inches wide, and 33 inches deep. They had a gabled shape with a hollow-box cross section. Although appearing to be heavy, they were of relatively light construction, with sides only 1-1/2 inches thick. In the central portion of the beams the only non-prestressed reinforcing was four intermediate-grade number-3 bars in the bottom, four in the top, and the 2 x 2 wire mesh which was used throughout. Further dimensions and details are shown in Figure 2. The theoretical section properties of the beams are given in Table I.

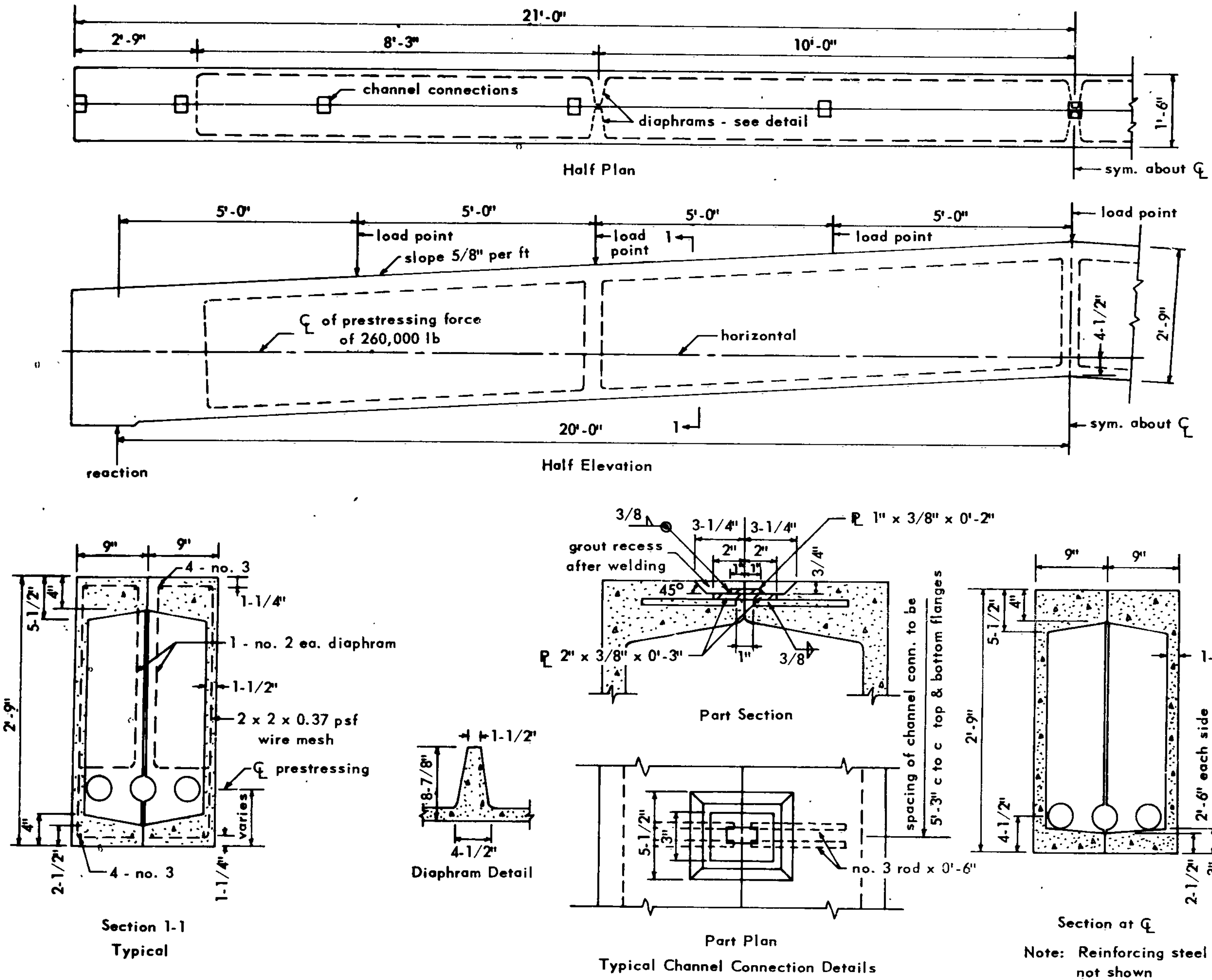



Figure 2. Drawing of test beams.

Table I. Theoretical Section Properties

| Physical Characteristics | Center Section | | Normal Section | |
|---|---|---|---|---|
| | Concrete Only | Transformed | Concrete Only | Transformed |
| Total depth of concrete section (in.) | 33.0 | 33.0 | 33.0 | 33.0 |
| Total width of concrete section (in.) | 18.0 | 18.0 | 18.0 | 18.0 |
| Area of entire concrete section ($in.^2$) | 211.5 | 219.9 | 219.0 | 227.4 |
| Distance to top fiber* (in.) | 14.71 | 14.78 | 15.22 | 15.26 |
| Distance to bottom fiber* (in.) | 18.29 | 18.22 | 17.78 | 17.74 |
| Moment of inertia* ($in.^4$) | 32,070 | 32,330 | 33,733 | 33,990 |
| Section modulus of top fiber* ($in.^3$) | 2,180 | 2,188 | 2,216 | 2,227 |
| Section modulus of bottom fiber* ($in.^3$) | 1,754 | 1,775 | 1,897 | 1,916 |
| Eccentricity of prestressing force* (in.) | 13.79 | 13.72 | varies | varies |

* Referenced to center of gravity of concrete or transformed sections.

A rich mix of concrete was used. It had a seven-day ultimate compressive strength of 6,680 psi when fog-cured. The high early strength was obtained with 9.44 bags of Victor Type III cement per yard and a moderate accelerator. The concrete had a water-cement ratio of 4.59 gallons per bag and an average slump of 2-1/2 inches.

The ends of each beam were reinforced with vertically placed intermediate-grade number-3 bars for a distance of 3-1/2 feet. The bars were bent and placed as shown in Figure 3.

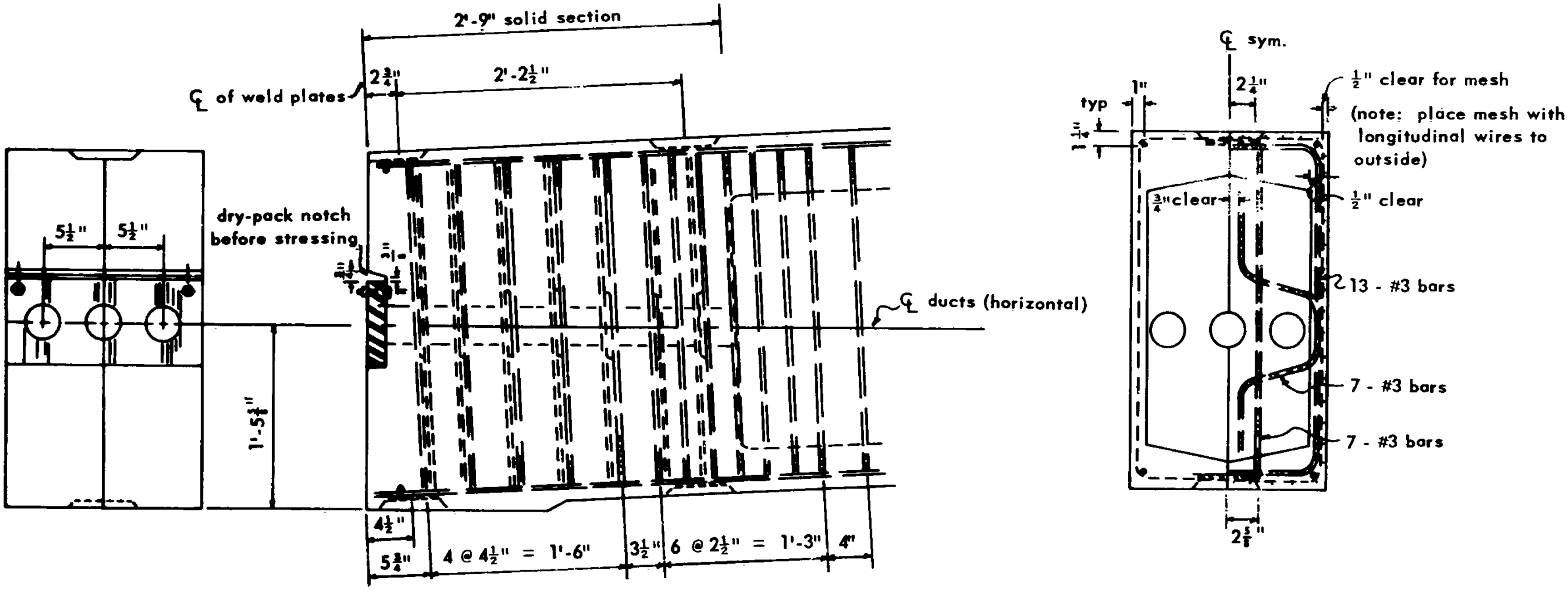
The three prestressing cables were straight. They were post-tensioned and unbonded. Each cable consisted of 12 wires of 1/4-inch diameter. These wires terminated with upset heads and were anchored by the C. D. Wailes Company system of prestressing, as shown in Figure 4. The initial prestressing force on a beam was 260 kips. This would be an initial stress of 148,300 psi, in each of the 36 wires, or about 0.70 of their yield strength. Further details of the properties of the prestressing steel and of the concrete are presented in Appendix A.

Due to the prestressing force and the dead load of the beam, the theoretical stresses at mid-span in the bottom and top fibers of the concrete would be respectively 2,880 psi compression and 190 psi tension based upon the theoretical section properties given in Table I.

The ultimate moment of the beams was calculated to be 936 foot-kips. They were under-reinforced, inasmuch as there was not sufficient prestressing steel and mild tensile steel to develop the ultimate strength of the concrete in the top of the box. This would be the normal condition for most unbonded prestressed beams. Such beams fail by the prestressing and mild tensile steel elongating and the beam deflecting until the neutral axis raises to within a few inches of the top surface. Eventually the concrete remaining above the neutral axis fails in compression.

The ultimate resisting moment is the product of the tensile force in the steel times the distance between this force and the equal and opposite compressive force in the concrete. An accurate calculation for the ultimate strength of unbonded beams is more difficult than for bonded ones, because the stress in the steel at rupture cannot be closely computed. In an unbonded beam the steel strain will be nearly constant between anchorages, and at mid-span the change in steel strain due to loadings will be considerably smaller than the strain in the concrete at the level of the steel.

For the hollow-box beams the ratio of change in steel strain to change in concrete strain at the same level at mid-span was determined experimentally to be 0.34 during the application of the external loads. This ratio is denoted by the letter F. For most unbonded prestressed beams the ratio varies from 0.1 to 0.3 and depends on conditions of loading and the shape of the unbonded tendons, while F is near 1.0 regardless of loading conditions for well-bonded tendons.[2,3,4]




Note: All reinforcing bars are #3.

Figure 3. End block reinforcing.

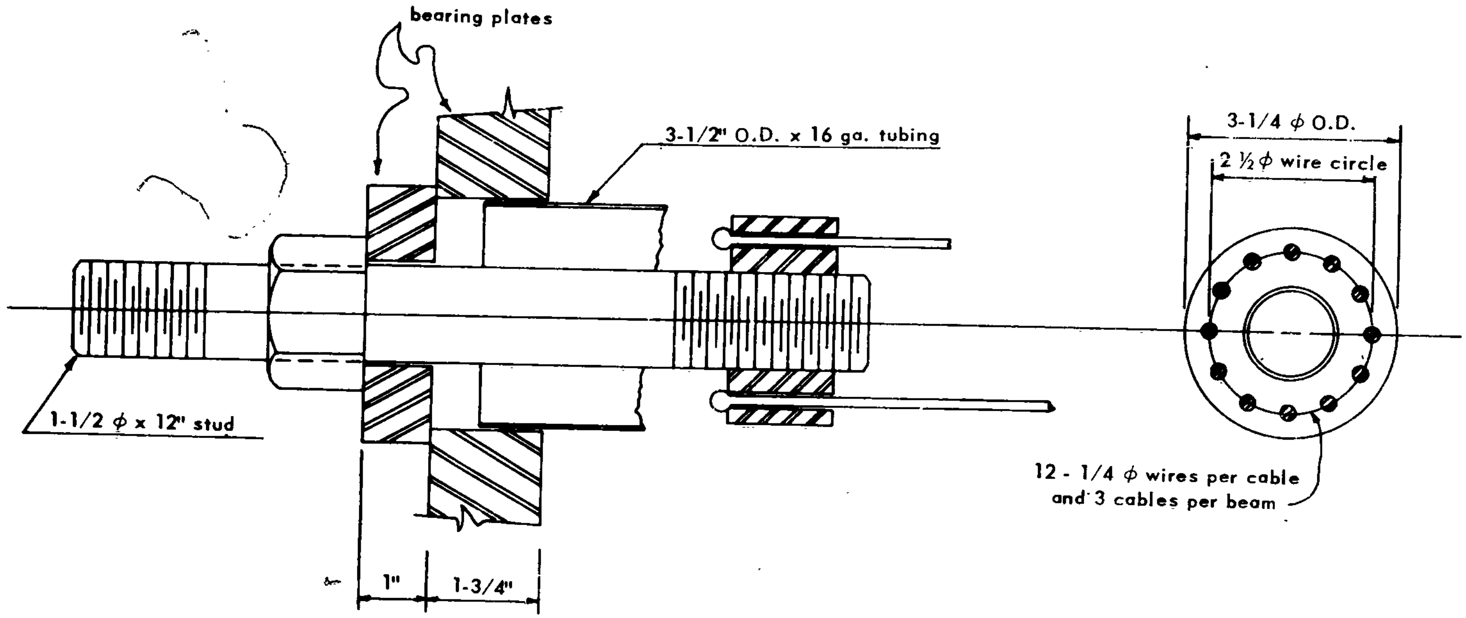



Figure 4. Prestressing end anchorage.

Knowing the ratio F, the following equation was used to calculate the stress in the prestressing steel at the ultimate moment for the hollow-box beams:

$$\frac{A_s\, f_{su}}{k_1 k_3\, b\, d\, f'_c} = \frac{F\, e_u}{e_{su} - e_{sp} + F\, e_u}$$

in which $A_s$ = area of tension steel = 1.768 in.$^2$ for prestressing steel and 0.51 in.$^2$ for mild tensile steel

$b$ = width of rectangular section = 18 in.

$d$ = depth to tension steel = 28.5 in. and 31.75 in.

$e_{su}$ = tensile strain in steel at ultimate moment

$e_{sp}$ = tensile strain in steel due to effective prestress = 0.00505 at completion of prestress

$e_u$ = ultimate compressive strain in concrete, assumed = 0.003

$f'_c$ = compressive strength of 6 x 12-in. field-cured cylinders = 7,800 psi at 28 days

$f_{su}$ = stress in reinforcement at ultimate moment

$k_1 k_3$ = coefficient determining average concrete stress in compression zone at ultimate moment, assumed = 0.83

Inasmuch as the stress-strain curve for the prestressing steel was nonlinear in the region under consideration, it was necessary to solve the equation by successive approximations. A value was assumed for the strain in the steel at the ultimate moment, $e_{su}$, and the stress, $f_{su}$, was computed. The stress-strain curve was then entered with the computed $f_{su}$ and a new value of $e_{su}$ obtained. By this method, $f_{su}$ was calculated to be 221,000 psi. The derivation of a similar equation is given in Reference 5 and will not be repeated in this report.

To determine the length of the moment arm it is necessary to establish the location of the centroid of the internal compressive force in the concrete at the ultimate moment. Its depth would be equivalent to $(k_2\, f_{su}\, A_s)/(k_1 k_3\, f'_c\, b)$, in which $k_2$ may be assumed equal to 0.42. The centroid was found to be about 1.5 inches from the top of the beam.

The ultimate moment at the completion of prestressing was calculated to be 221,000 (1.768) (28.5 - 1.5) + 45,000 (0.51) (31.75 - 1.5) = 11,230 inch-kips, or 936 foot-kips.

It is interesting to note that the theoretical ultimate moment after four years is within 1 percent of the above value. The loss of prestress with time is apparently compensated for by the increase in the compressive strength of the concrete.

## Test Procedure

Eight identical beams were tested under four different loading conditions as shown in Table II. The indicated loads were divided equally between seven load points spaced 5 feet apart. The theoretical moments corresponding to the applied loads are shown graphically in Figure 5, and can be compared with the computed ultimate moment. Each pair of beams provided a check upon one another and increased the reliability of the results.

Table II. Applied Loads

| Beams | Loading* |
|---|---|
| 1 and 2 | 0 |
| 3 and 4 | 35 kips |
| 5 and 6 | 56 kips for 6 months and 35 kips thereafter |
| 7 and 8 | 77 kips for 30 days and 35 kips thereafter |

* Loads in addition to dead load and prestressing force.

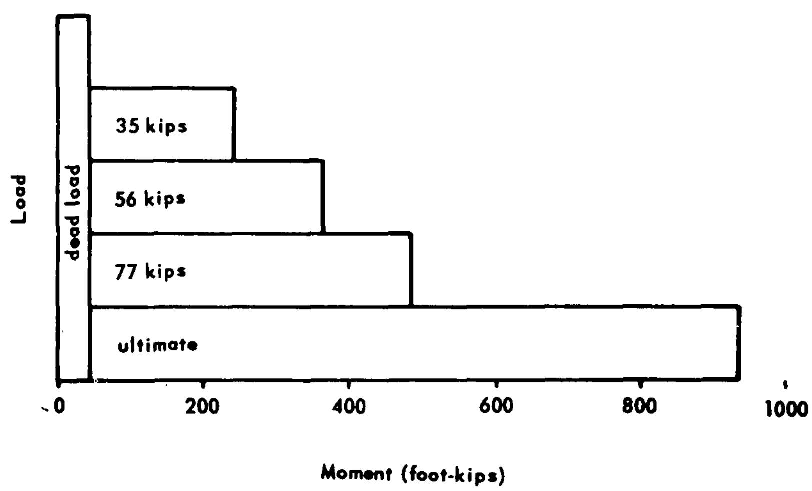



Figure 5. Theoretical moments corresponding to given loads.

The beams were cast and prestressed at NCEL. Details of the fabrication are presented in Appendix B. As the beams were completed they were prestressed, placed on simple supports on a 40-foot span, and the live load applied.

The supports consisted of a rocker 1 foot from one end, and a rocker-roller combination 1 foot from the other end. The loads consisted of especially cast concrete weights suspended from a harness arrangement that applied the load through a rocker. The general test setup can be seen in Figure 1. Prior to loading, the concrete weights were moved into position and the beam placed over them. The harnesses were then secured to the weights and used to jack them up so that they were suspended from the beam.

Where the load was to be reduced after a period of time, the weights were made in two parts so that the lower part could easily be removed.

## Instrumentation

Deflections, prestressing forces, and concrete strains were measured periodically.

For measuring deflections, three brass reference points were set in the top surface of each beam. There was one over either end support and one in the middle. Deflections were read with an engineer's level and a special rod. The rod had 1/16-inch graduations and leveling bubbles for vertical alignment.

The prestressing forces were measured with SR-4 strain gages and a Baldwin Type M Indicator. A solid bar 14 inches long, referred to as a coupler, was inserted into each of the prestressing cables. Four SR-4 electrical-resistance strain gages, Type AB-7, were so arranged on this coupler as to form a complete Wheatstone bridge with all arms active. They were cemented on with Armstrong Type A-1 epon cement. The gages were wired, waterproofed, and calibrated against load in a standard testing machine before being placed into the prestressing cables. Two completed installations are shown in Figure 6.

Concrete strains were measured with Carlson gages and test set. These gages are of the unbonded electrical-resistance type comprising two active arms of a Wheatstone bridge. The bridge was completed, balanced, and the strain indicated by the test set. This instrument was also used to measure the total resistance of the two active arms. From this reading the temperature of the gage and surrounding concrete could be calculated. Figure 7 shows a Carlson gage being installed. The gage was held by the concrete gripping the raised lip on either end, thus giving an effective gage length of 6 inches.

Eight strain gages were used per beam. They were placed 1-3/4 inches from, and parallel to, either the top or bottom surfaces of the concrete. Four of the gages were placed 1 foot from the center of the beam, and the other four 4-1/2 feet from one end as shown in Figure 8.

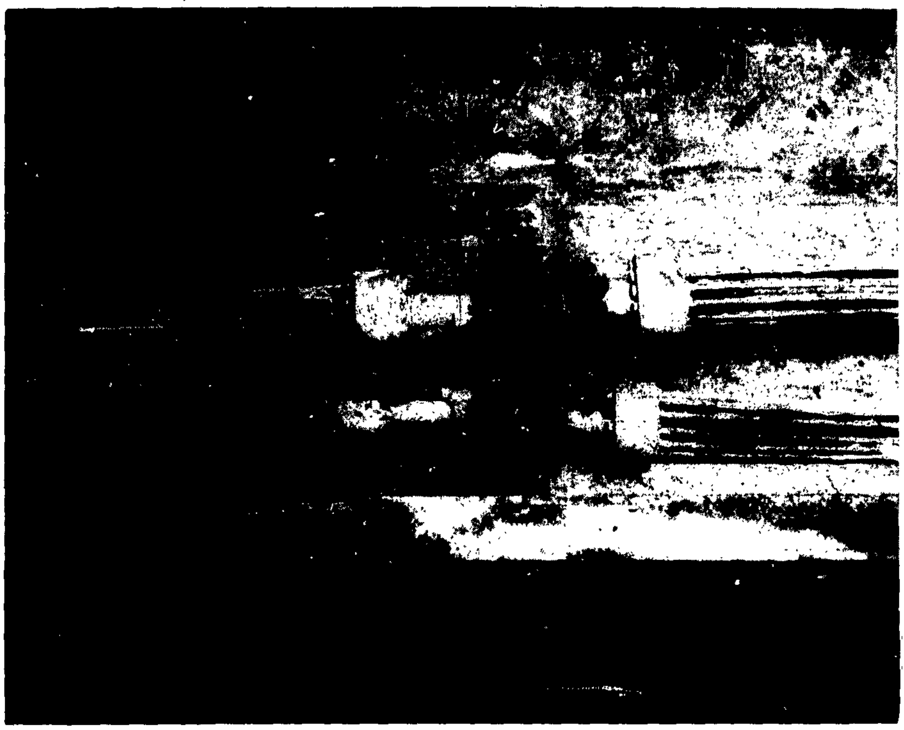


Figure 6. Two prestressing-force gages (couplers) in place.

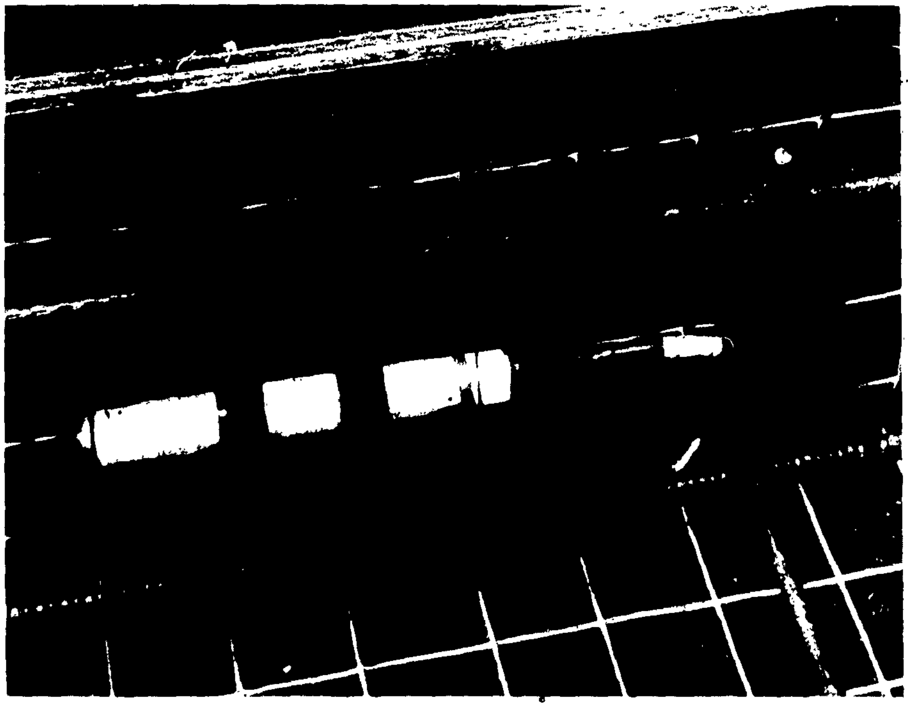


Figure 7. Carlson strain gage in place.

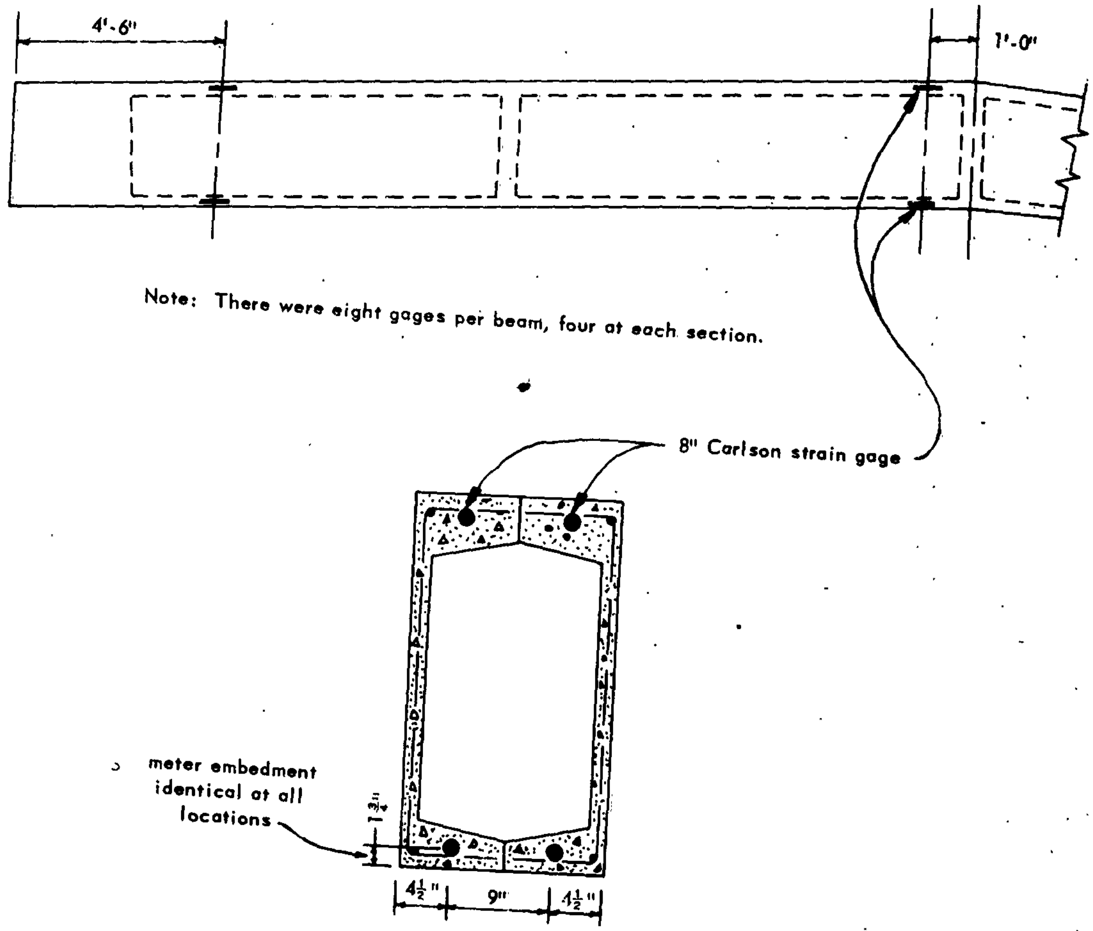



Figure 8. Location of Carlson strain gages.

## RESULTS AND DISCUSSION

The data obtained from the measurements of deflections, concrete strains, and prestressing forces were reduced and plotted. The results of a macroscopic examination of the beams for any cracks or defects were noted.

### Deflections

All deflection measurements were made relative to an initial zero reading which was taken as the beams lay on their sides just prior to prestressing. After the completion of the prestressing, and with the beams upright and the full dead load acting, they had an average upward deflection of almost 1/2 inch. The initial deflections for each beam are shown in Figure 9. Beam number 1 was omitted because of a questionable zero reference. All subsequent deflections of this beam compared closely to beam number 2. There were, however, some differences in the response of the various beams to the prestressing and dead loads, and between pairs of beams to the applied live load. These differences may be observed in Figure 9.

During the first day following prestressing all the beams continued to deflect upward. The average increase was about 1/8 inch. For the following 28 days there was no significant change in the deflection of beams 1 and 2. These beams had no additional load applied to them.

The day following prestressing the other six beams were loaded. Beams 3 and 4 each received 35 kips and deflected about 3/8 inch, leaving a residual upward deflection of about 3/16 inch. The 56-kip load applied to beams 5 and 6 caused them to have a slight residual deflection downward. Beams 7 and 8 were loaded with 77 kips and had an average residual downward deflection of 1/2 inch.

During the following 28 days all of the loaded beams continued to deflect downward. Their deflections during this period were approximately as large as the deflections due to loading. The beams with the 35-kip loads had an additional average deflection 1.1 times the immediate deflection due to loading, and the beams with the larger loads had additional average deflections 0.8 times their loading deflections. t was at the end of this period that the largest deflection was recorded. Beam 7 h.d a deflection of almost 1-1/2 inches. It was equivalent to L/327.

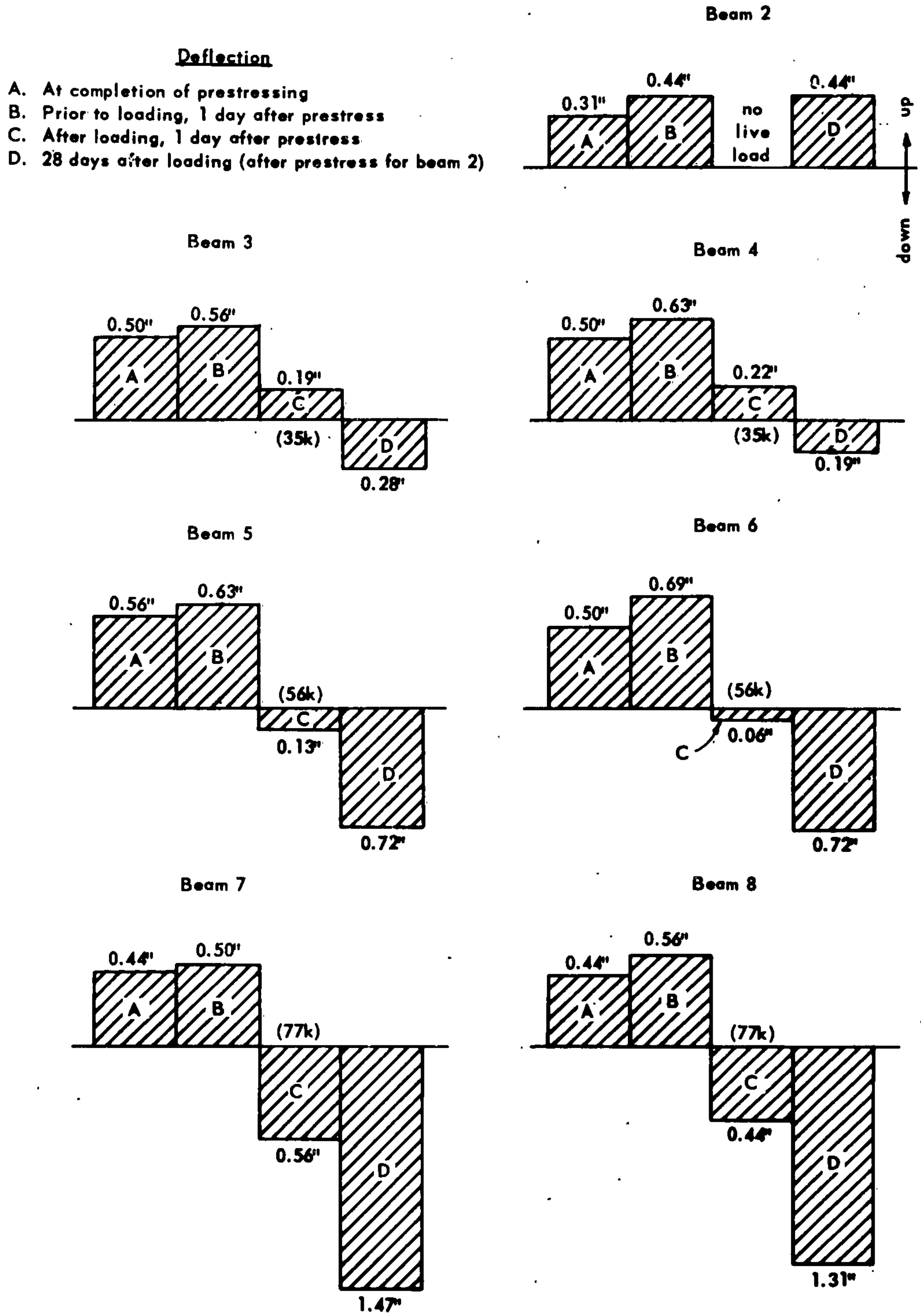



Figure 9. Cumulative deflections during first 29 days after prestressing.

If the load had been left on beams 7 and 8 they would probably have continued to show additional deflection. At this time, however, the live loads on these beams were reduced from 77 to 35 kips, a reduction of 55 percent. Deflection measurements showed that the two beams immediately recovered about 55 percent of their initial deflection due to loading. There was some additional recovery with time, as can be seen in Figure 10, but this was only slight.

After being applied for six months the loads on beams 5 and 6 were reduced from 56 to 35 kips. There was a corresponding elastic recovery of the initial deflection due to loading. The full recovery of immediate load deflections upon reduction of the load was also observed by W. S. Cottingham and others on prismatic-shaped prestressed sections.[6]

After one year there was not much change in the deflection of any of the hollow-box beams. The two unloaded ones showed a trend to continue deflecting slightly upward, and the six with 35-kip loads showed a trend to continue deflecting slightly downward. In over three years, however, the average change for all of the beams was only about 1/16 inch.

The beams were located outdoors and subjected to existing weather conditions, which accounts for the cyclic changes observed in the data. No attempt was made to draw a curve through all of the points shown in Figure 10. Instead, straight lines were fitted to the data obtained after one year by the method of least squares, thus best showing the trend.

Both beams 5 and 6 (56 kips initially) and beams 7 and 8 (77 kips initially) had the same deflections after the first year. Beams 5 and 6 must have experienced the same inelastic deformations during the initial six-month period as beams 7 and 8 did during the initial one-month period before the loads were reduced to 35 kips. As could be expected, all of these beams exhibited greater final deflections than the beams that were continuously loaded with 35 kips.

Strains and Stresses

All strains and corresponding stresses reported herein are 1-3/4 inches from the top or bottom fibers of the concrete, as shown in Figure 8. Strains have been converted to stresses only during prestressing and loading. At other times the variation of strain due to other causes and the variation of the elastic modulus are so comparatively large as to render any conversion to stress quite illusory.

The stresses due to prestressing and loading are shown in Figures 11 and 12. These values were arrived at by averaging the change in strains in corresponding positions on each pair of beams and multiplying by $4 \times 10^6$ psi, the average static modulus of elasticity of the field-cured cylinders (Appendix A). Figure 11 shows the stresses at a section 1 foot from the center of the beams. At 1-3/4 inches from the bottom the average stress due to the prestressing force and the dead load of the beam was 3,090 psi. This was the highest stress obtained and was 0.41 of the average ultimate strength of the concrete at that time. The prestressing force caused the top of the beams to go slightly into tension. The 77-kip live load on beams 7 and 8 caused the bottom to have approximately zero stress and the top gages to record an average of 2,480 psi.

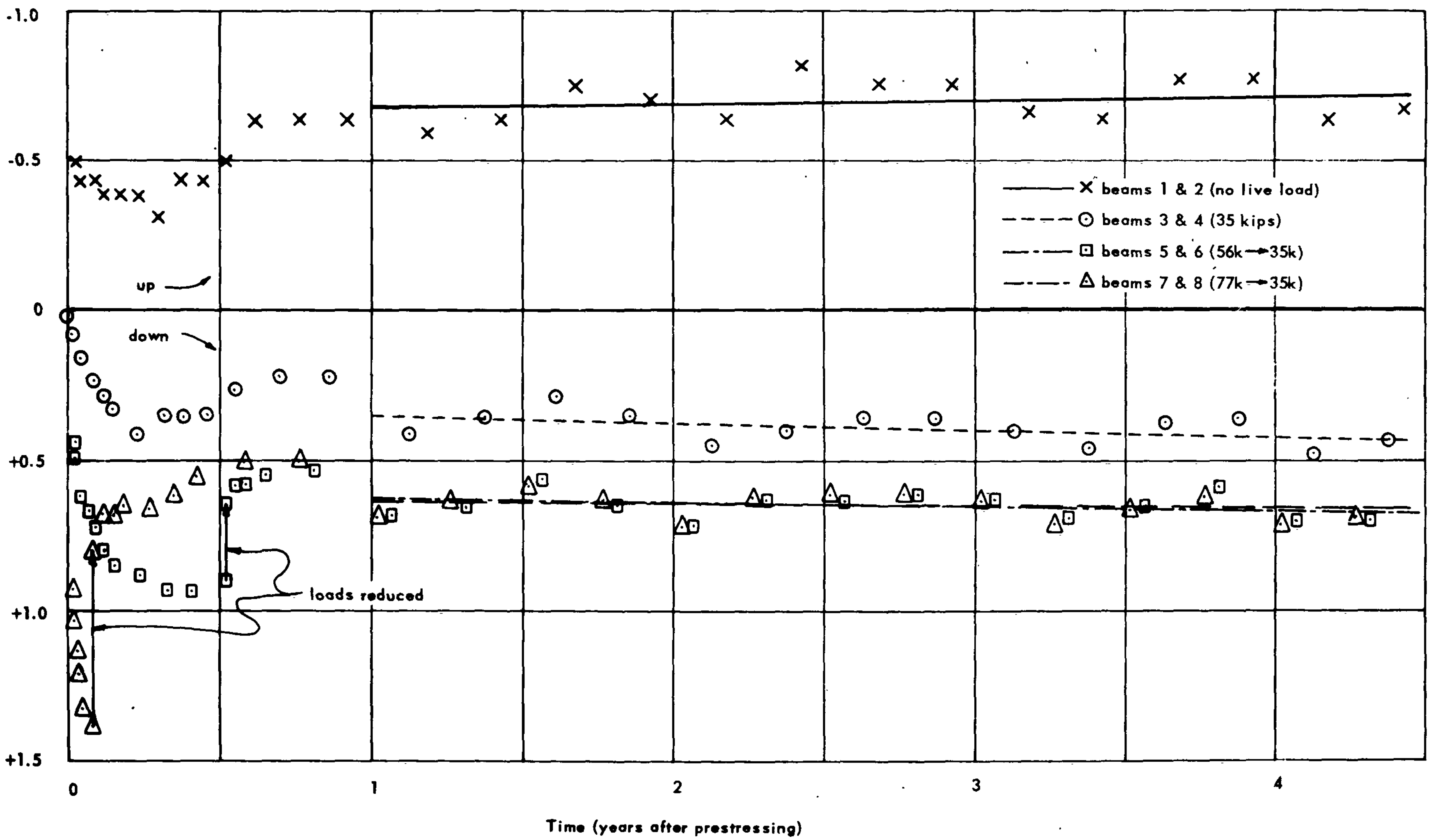



Figure 10. Deflection at center of beams versus time.

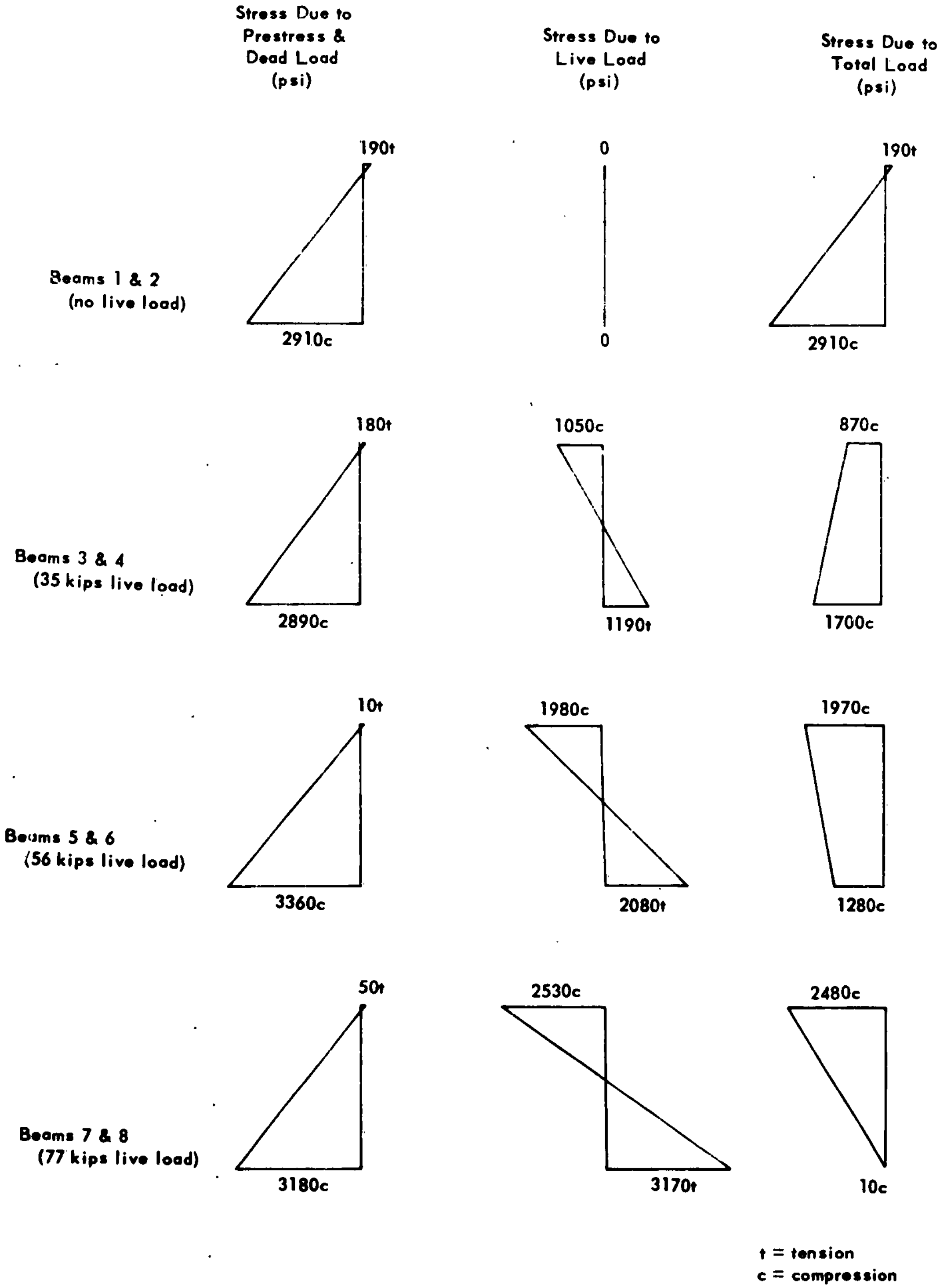



Figure 11. Stresses at 1 foot from center section due to prestressing and loading.

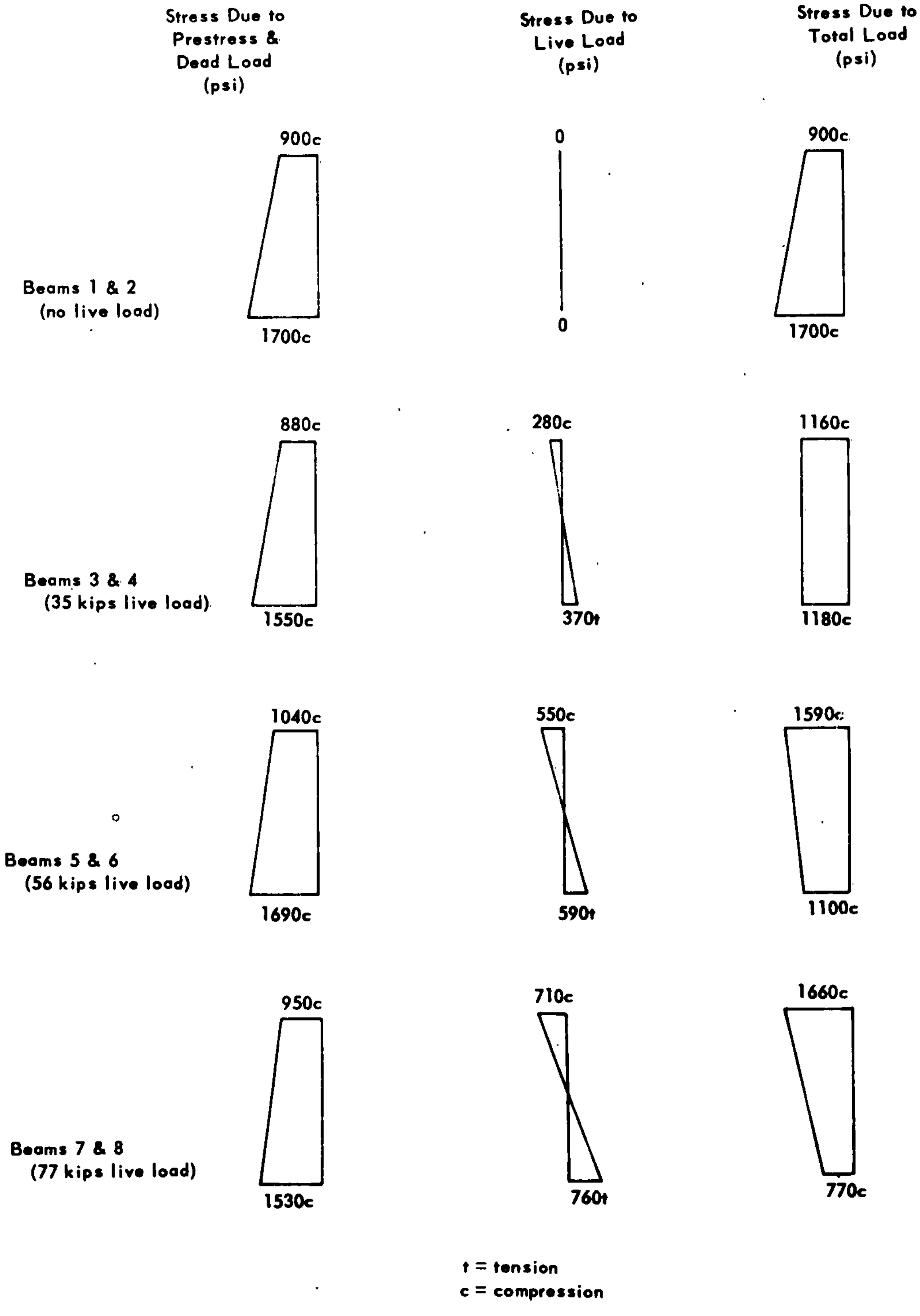



Figure 12. Stresses at 4 feet 6 inches from end due to prestressing and loading.

After the 77-kip and the 56-kip loads were reduced to 35 kips, all of the loaded beams had similar stress distributions. There was almost twice as much compressive stress in the bottom as in the top.

Figure 12 shows that all of the stresses 4 feet 6 inches from the end were compressive. The highest was 1,700 psi and the lowest 770 psi.

The variations of strain with time are shown in Figures 13 through 16. The strain at a given location is plotted on the same graph for all of the beams, thus showing the comparable effects of the different initial loads. Since there were two sections with gages located near the top and bottom of each, there are four graphs. The strain data were averaged and corrected for shrinkage before being plotted. For these graphs, therefore, the strain was considered to be zero just prior to prestressing.

The strains due to prestressing and loading of the beams have been discussed in terms of their corresponding stresses.

During the 28 days following loading, the gages near the bottom center of the beams showed elongations of 137 microinches per inch for beams 3 and 4, 197 for beams 5 and 6, and 321 for beams 7 and 8. This was in spite of the 1,700-psi, 1,280-psi, and 10-psi compressive stresses in these respective areas. This phenomenon of concrete elongating under compressive loads was most probably due to two causes. One of these would be the 6 to 7 percent loss of prestress that occurred during this period. This would account for about 50 microinches per inch. The balance of the elongation may be attributed to the deferred strain following the reduction of compressive stress in the bottom of the beams due to the application of the loads.

Compressive stresses were substantially reduced in the top center of beams 5 through 8 when the live loads were partially removed. The phenomenon of concrete having time-dependent elongations under compressive stresses was observable during the following several months.

The discontinuities in the strain data between 6 and 9 months are due to the changes brought about by the wetting of the beams by rain. This was the first rain to which the beams were subjected since they were cast, and followed a very dry period. The effect of precipitation on the beams is shown in Figure 23 in Appendix A.

Figures 13 and 15 also show that after one year the concrete in the bottom of the beams gradually contracted. The only way that the beams could have shown a trend to continue deflecting slightly downward during this period, as previously noted, would be for the top fibers to contract more than the bottom fibers. This trend can be observed in the strain data.

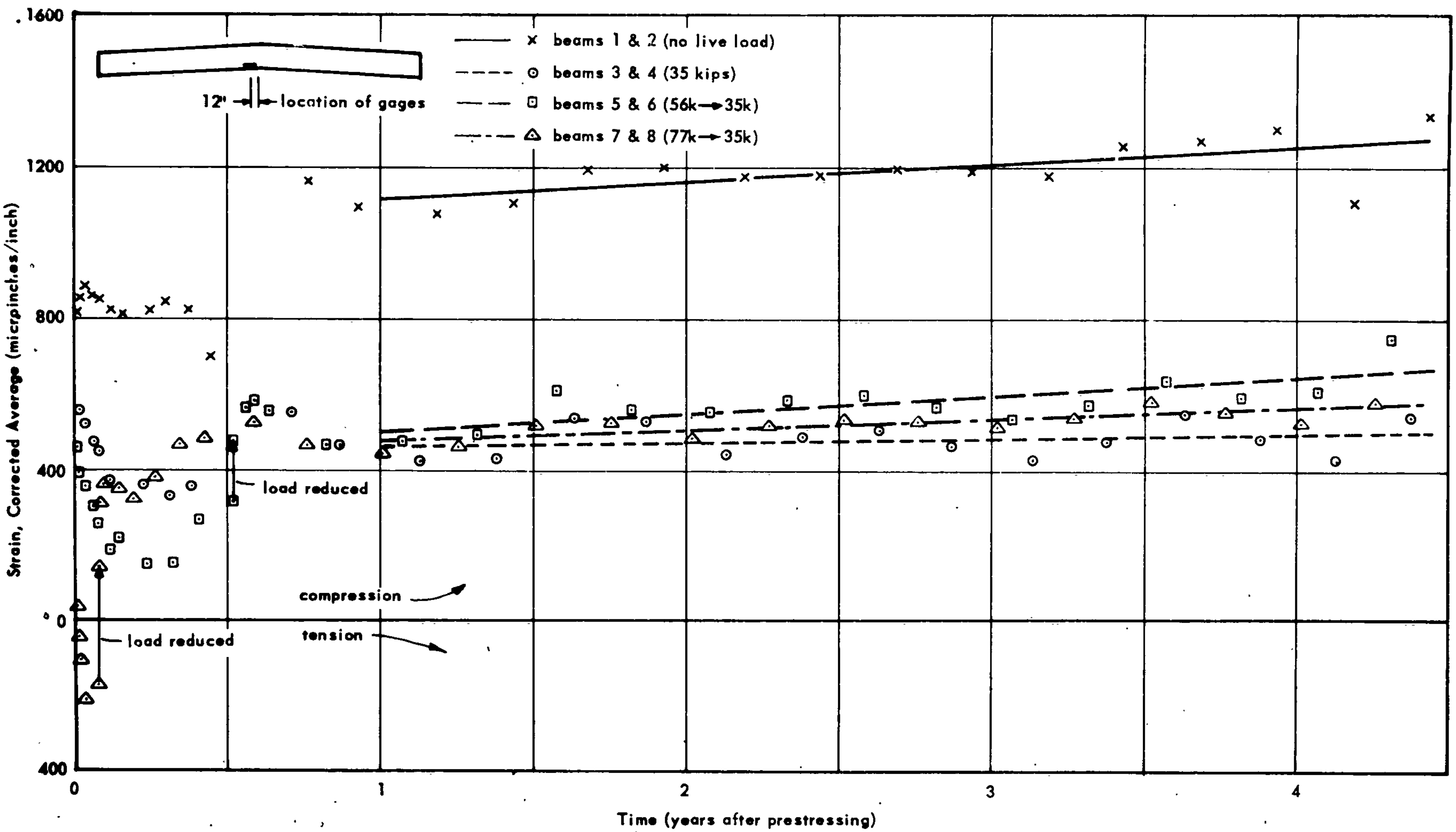



Figure 13. Corrected average strain near bottom center of beams versus time.

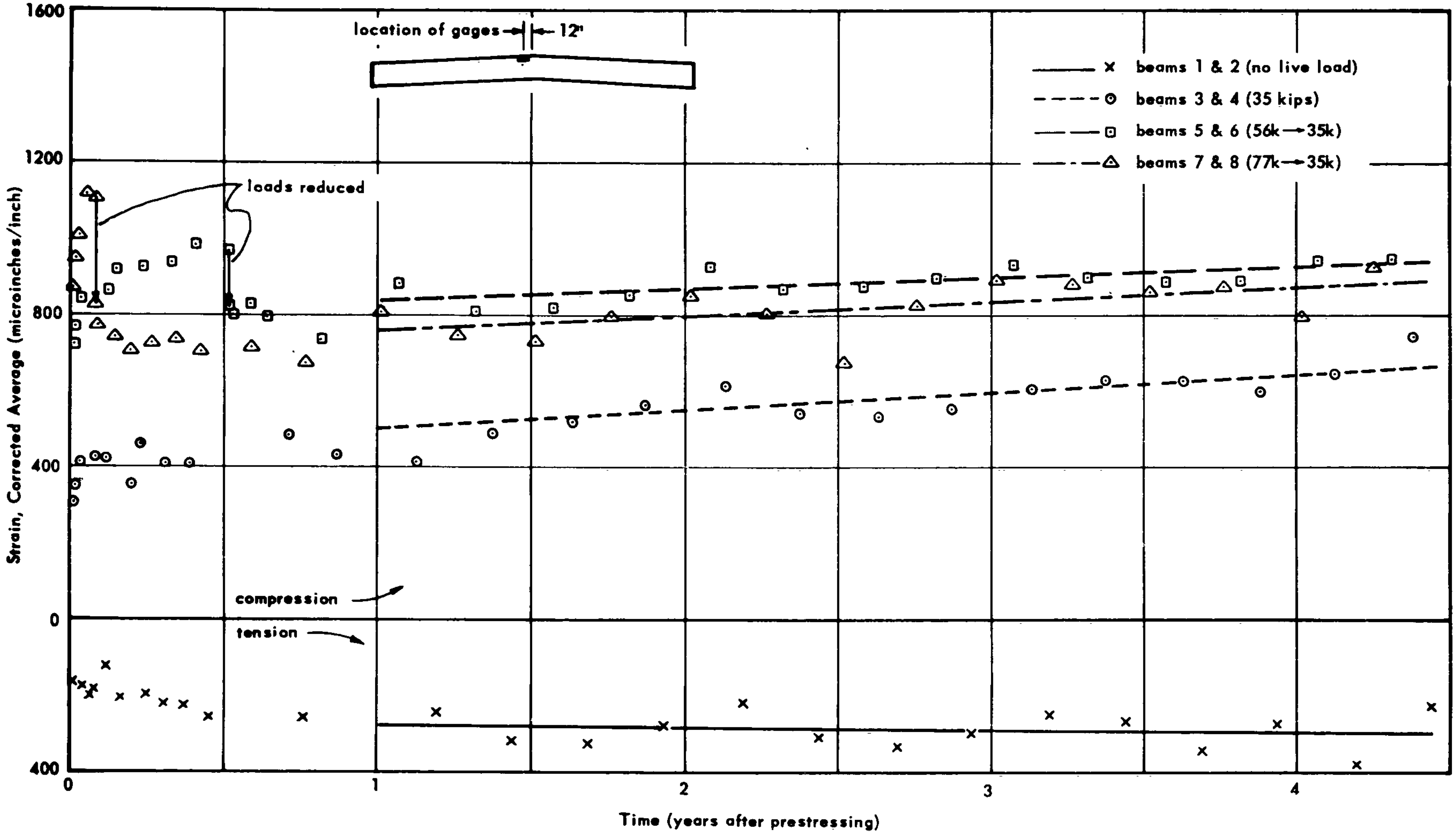



Figure 14. Corrected average strain near top center of beams versus time.

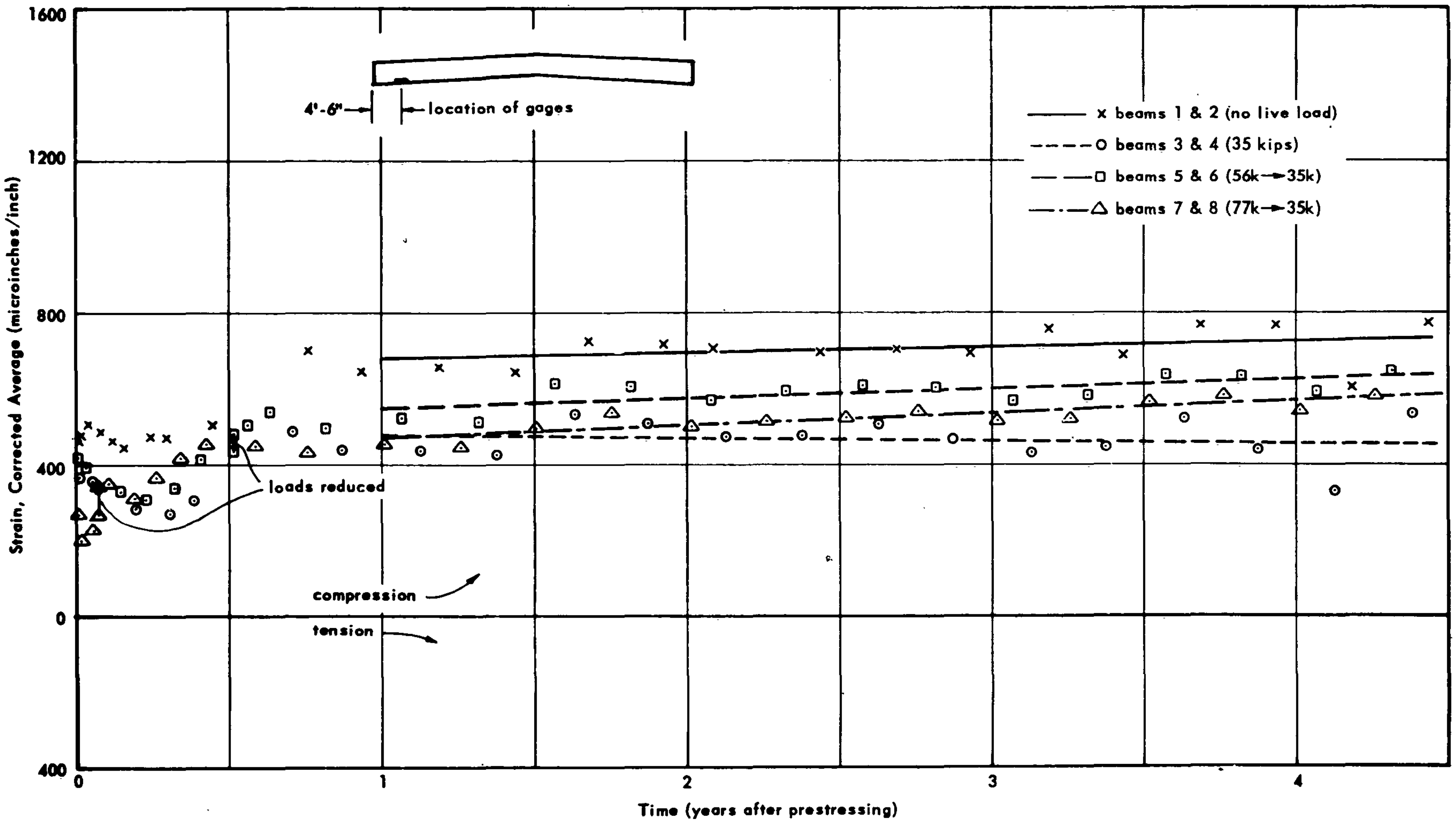



Figure 15. Corrected average strain near bottom end of beams versus time.

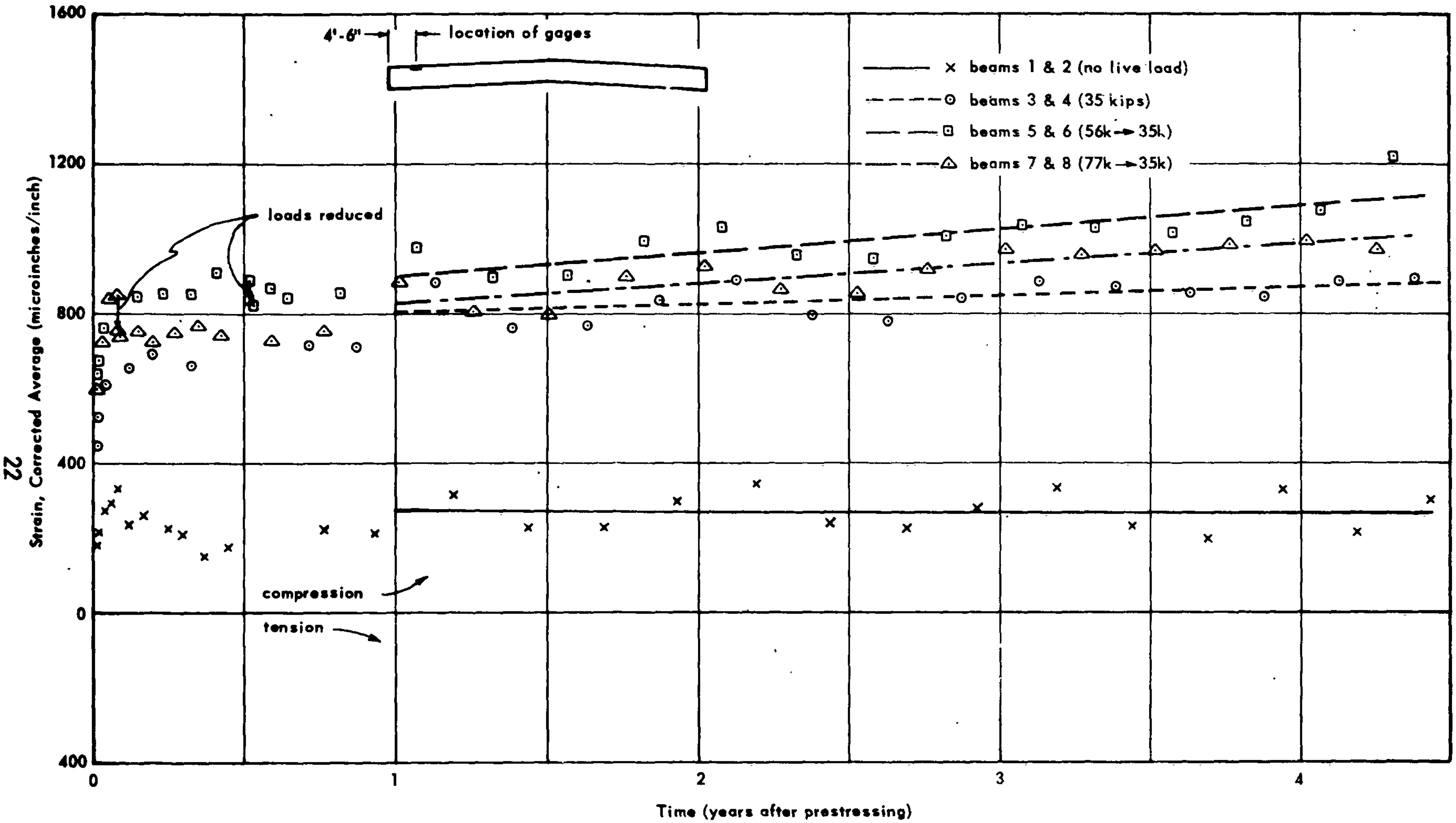



Figure 16. Corrected average strain near top end of beams versus time.

## Prestressing Forces

All of the beams were initially prestressed to 260 kips. The manner in which this prestressing force changed with time and load variations is shown in Figure 17. The plotted values are the actual prestressing forces at the times indicated, including increases or losses due to load changes.

The unloaded beams experienced more loss than any of the beams which had been loaded. This may have been due in part to the stress distribution in the unloaded beams. They had much higher compressive stresses in the bottom.

Figure 18 shows the percent of the initial prestressing force lost with time. Only the time-dependent changes are included; changes due to loading and unloading are omitted. The arithmetic mean for each pair of beams is plotted. Overall, the difference between beams was not great; the average difference between percentages of prestress lost was 0.8 percent. During the first year about 13 percent was lost. In the following 3-1/2 years an additional 9 percent was lost, or a total of about 22 percent in 4-1/2 years.

After the first year, the effects of loss of prestress on deflections (Figure 10) and on strains (Figures 13 through 16) were very slight. The initial deflection due to prestressing was 1/2 inch. Thus, a 9-percent change would be less than 0.05 inch. The change of strain in the bottom of the beams would be less than 80 microinches per inch, and there would be essentially no change in the top of the beams at mid-span.

The prestress loss reported herein is the total, and the following may be contributing factors:

1. Time-dependent losses in the concrete (these are the summation of creep and shrinkage)

2. Creep of the prestressing steel

3. Plastic flow of the steel in and around the anchorages

## Cracks

A macroscopic examination was made of the beams at the time the report was written. Most of the beams appeared to be in good condition. The only cracks observed were in beams 1 and 5. There was a vertical crack in the center of beam 1, halfway across the top and most of the way down one side. Cracks in the top could be expected for this beam, inasmuch as it had no live load and the prestressing produced tensile forces in that area. Also, one side of beam 1 contained numerous small horizontal cracks 1 to 3 feet long throughout its length.

The only other cracks were on one side of beam number 5. There were eight vertical hairline cracks starting at the top of the beam and running to within a few inches of the bottom. The cracks were about equally distributed between one of the supports and mid-span.

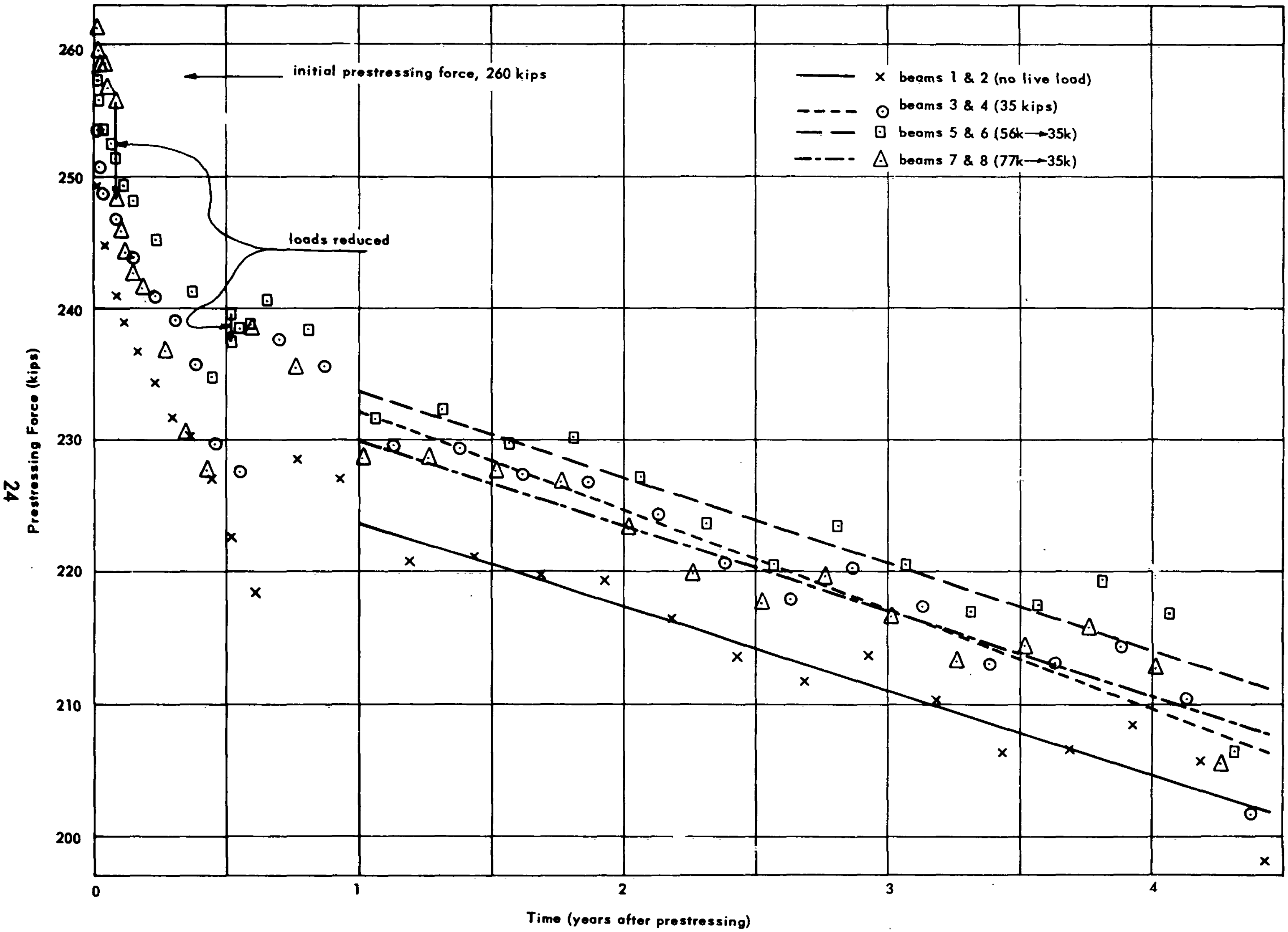



Figure 17. Prestressing force versus time.

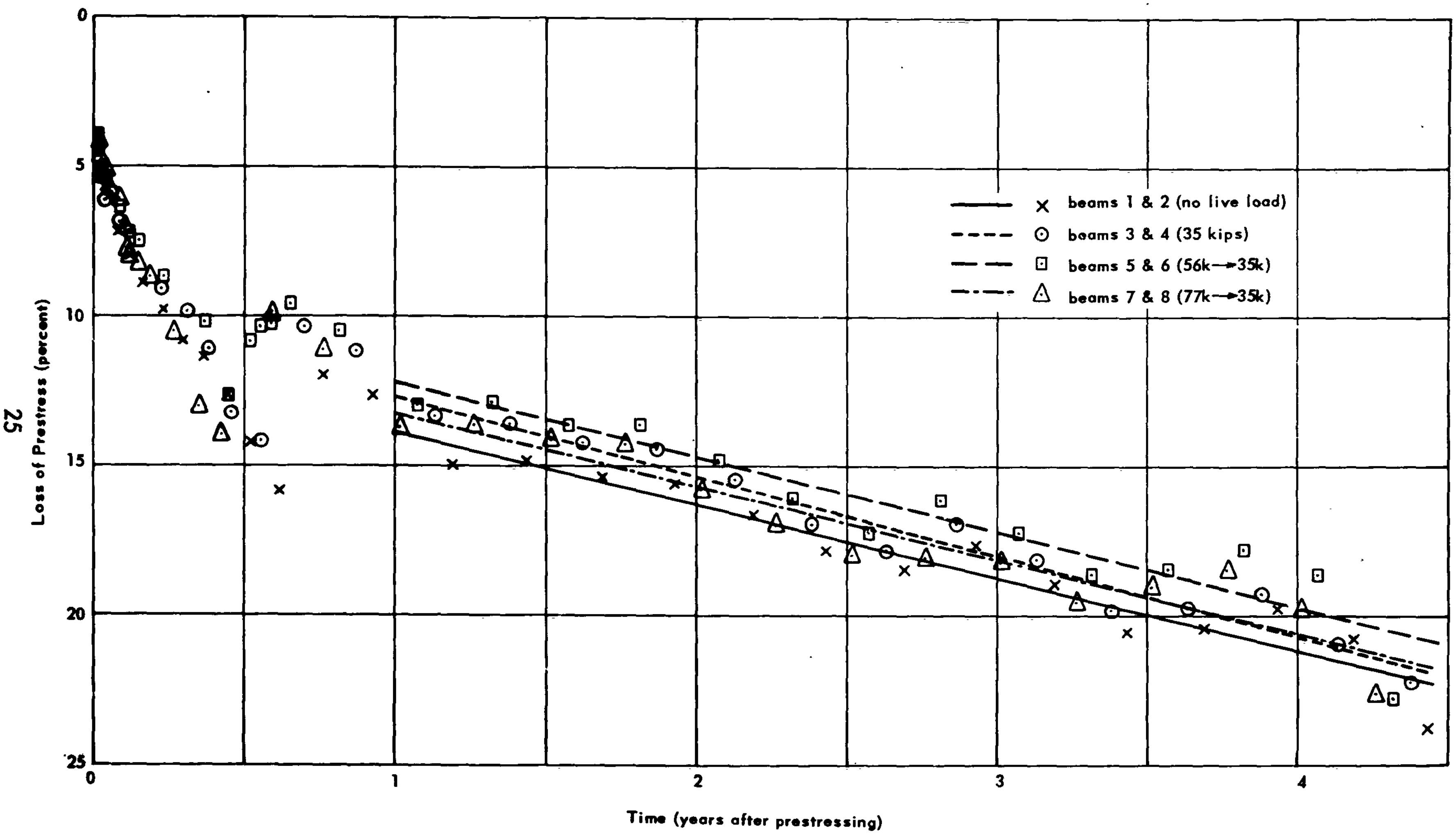



Figure 18. Loss of prestress versus time.

## CONCLUSIONS

Based upon the results of long-time tests of eight hollow-box beams, it may be concluded for this type of beam that:

### Deflections

1. Any significant live load may produce an additional time-dependent deflection during the 28 days following loading approximately equal to the immediate elastic deflection due to application of the load.

2. If the mid-span stress distribution is such that there is about twice as much compression in the bottom as in the top fiber, the maximum deflection will be approached during the first year.

3. If there is no live load, time-dependent deflections are upward. Most of the deflection takes place during the first year. There is, however, a tendency for unloaded beams to continue deflecting upward for a number of years.

4. Time-dependent deflections are for the most part permanent. Upon reducing the load, there is only a slight (roughly 15 percent) recovery of such deflections with time.

5. Deflections at the time of loading are completely recovered when the load is removed, and are thus elastic.

### Strains and Stresses

6. Time-dependent strains (shrinkage, creep, etc.) are greater than the strains due to normal working stresses (prestressing and loading).

7. The previous load history of concrete is important in determining its response to presently applied loads. Following a sizable reduction of a compressive stress, concrete can have time-dependent elongations even though it is still under the remaining compressive stress.

8. The relative strains, and not the absolute strains, determine the direction in which a beam will deflect. The beams deflected downward with increasing contraction of the bottom fiber when the top fiber contracted even more.

### Prestressing Forces

9. Unloaded beams experience more loss of prestress than loaded beams; although after one year the rate of loss is about the same for loaded and unloaded beams.

10. During the first year prestressing losses can be as high as 13 percent, and in 4-1/2 years they can be as high as 22 percent.

11. After the first year, and up to 4-1/2 years, loss of prestress does not significantly effect deflections, strains, or ultimate strength.

ACKNOWLEDGMENTS

The beams were designed and the loadings designated by the Bureau of Yards and Docks, Code E-202C. The technical contributions of Mr. C. K. Wiehle, a co-worker during the early phases of this task, and the guidance rendered by Mr. S. L. Bugg, Director of the Structures Division, are gratefully acknowledged.

REFERENCES

1. Lin, T. Y. Design of Prestressed Concrete Structures. John Wiley and Sons, New York, 1955.

2. Feldman, A. "Bonded and Unbonded Prestressed Concrete Beams Failing in Flexure." MS thesis, Civil Engineering Department, University of Illinois, 1954.

3. Baker, A. L. L. "Further Research in Reinforced Concrete, and Its Application to Ultimate Load Design." Proceedings, Institution of Civil Engineers, London, Vol. 2, Part III (August 1953), pp. 269-310.

4. Gifford, F. W. "The Design of Simply Supported Prestressed Concrete Beams for Ultimate Loads." Proceedings, Institution of Civil Engineers, London, Vol. 3, Part III (April 1954), pp. 125-143.

5. Janney, J. R., E. Hognestad, and D. McHenry. "Ultimate Flexural Strength of Prestressed and Conventionally Reinforced Concrete Beams." Journal of ACI, Vol. 27, No. 6 (February 1956), p. 601.

6. Cottingham, W. S., P. G. Fluek, and G. W. Washa. "Creep of Prestressed Concrete Beams." Journal of ACI, Vol. 32, No. 8 (February 1961), p. 929.

## Appendix A

## PROPERTIES OF MATERIALS

### PRESTRESSING STEEL

The 1/4-inch-diameter steel wires used for prestressing were tested to determine their modulus of elasticity, proportional limit, yield strength, and ultimate strength. Eight specimens, 2-1/2 to 3 feet long, were tested at a loading rate of 30,000 psi per minute. This rate was used throughout the test except near the yield strength, where it was somewhat slower. The results of these tests are presented in Table III.

Table III. Properties of Prestressing Steel

| | Arithmetic Mean (psi) | Standard Deviation (psi) | Coef. of Variation (percent) |
|---|---|---|---|
| Modulus of Elasticity | $29.1 \times 10^6$ | $0.18 \times 10^6$ | 0.6 |
| Proportional Limit | 139,000 | 10,700 | 7.7 |
| Yield Strength | 214,000 | 1,200 | 0.6 |
| Ultimate Strength | 239,000 | 2,900 | 1.2 |

The proportional limit was taken as that point where the stress-strain curve begins to deviate from a straight line. Inasmuch as this is a gradual process and the exact location of the point somewhat questionable, this property therefore shows the greatest coefficient of variation. The coefficient of variation of the other properties was very small. The yield strength was determined by the 0.2-percent offset method. A typical load-strain curve is shown in Figure 19.

All of the test specimens failed within 1/8 inch of the grip. To evaluate the effect on the ultimate strength of upsetting the ends of the wires, half of the specimens had their ends upset in the same manner as those in the beams. The load was then applied to the test wires through the upset ends. The other half of the specimens were gripped for testing with Strandvises. The average ultimate strengths obtained by these two methods of holding the test wires were within 1/2 percent of each other, and all eight tests were averaged to obtain the results reported in Table III.

### CONCRETE

The pertinent physical and mechanical properties of the hardened concrete and of the materials used in the concrete mix were carefully tested in accordance with ASTM procedures. Any deviations from these procedures have been noted.

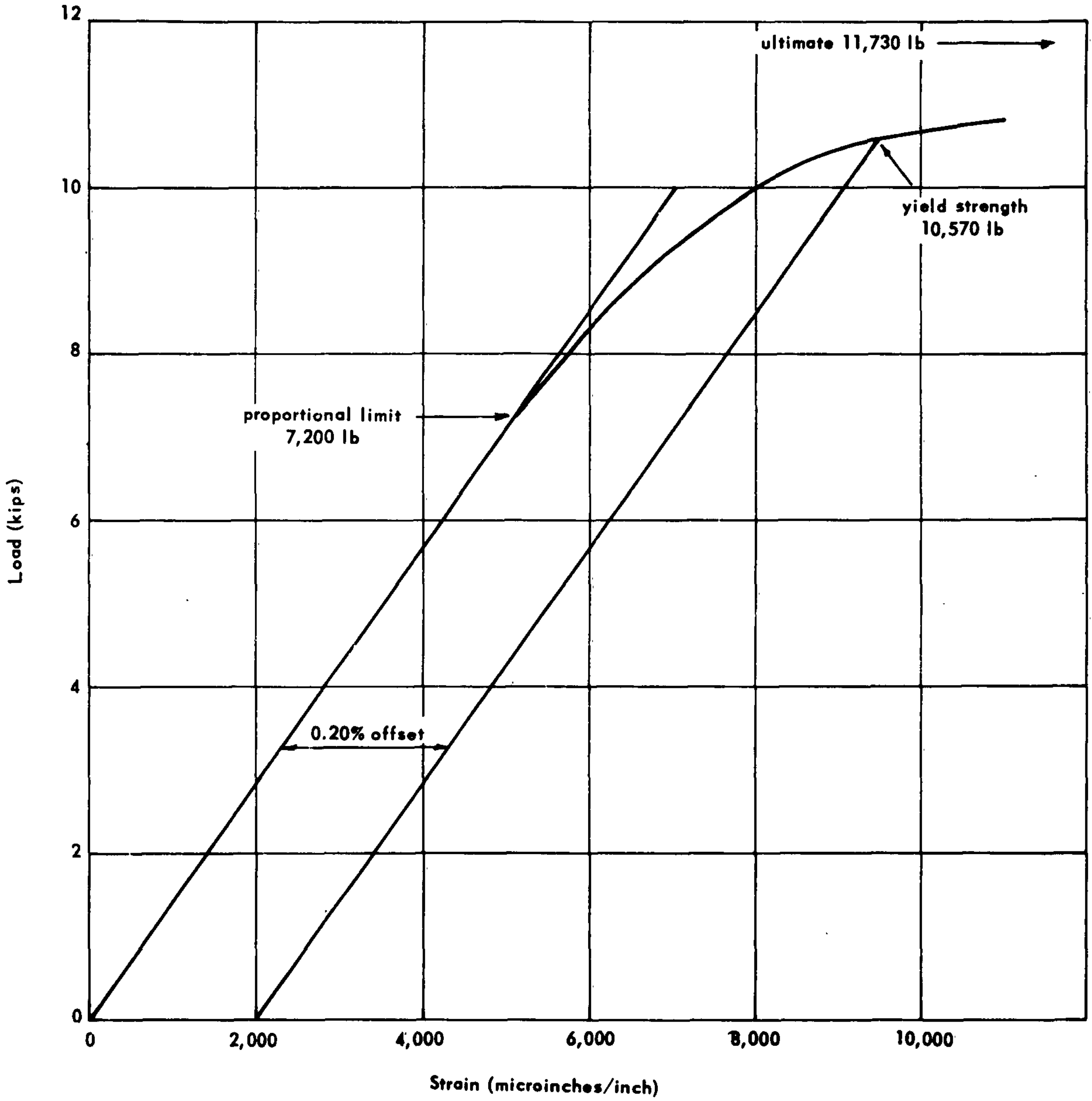



Figure 19. Typical load-strain curve for prestressing wires.

## Aggregates

The aggregates were obtained from local sources. The coarse aggregate was pea gravel from Grimes Canyon in California. It had a saturated-surface-dry bulk specific gravity of 2.60, an absorption of 1.1 percent, and an average total moisture content of 1.6 percent. The fine aggregate was a sand from the Santa Clara Somis Pit in California. It had a saturated-surface-dry bulk specific gravity of 2.58, an absorption of 2.1 percent, an average total moisture content of 5.0 percent, and a fineness modulus of 3.09. The results of sieve analyses on the aggregates are shown in Figure 20. The aggregates were combined by mixing 51 percent of the fine and 49 percent of the coarse by weight.

## Hardened Concrete

To determine the properties of the hardened concrete, a minimum of twelve 6 x 12-inch cylinders, five 6 x 6 x 24-inch beams, and two 2 x 2 x 10-inch prisms were taken during every casting operation. The cylinders were tested for density, sonic and static moduli of elasticity, and ultimate compressive strength. The beams were used to determine modulus of rupture, and the prisms were used to determine coefficient of expansion, and shrinkage.

Density. Volume and weight measurements on the cylinders showed the average density to be 146 pounds per cubic foot.

Moduli of Elasticity. The static modulus of elasticity was determined after the cylinder had first been loaded to 1,000 psi three times. The modulus was then based upon the axial strain induced by raising the compressive stress from 350 to 1,000 psi. This was done twice and the average of these values recorded. The static and sonic moduli of elasticity are shown in Table IV for the fog-cured cylinders. The coefficient of variation of these moduli for the seven-day tests was 5.3 and 3.6 percent respectively.

Table IV. Results from Fog-Cured Cylinders

| Age (days) | Number of Specimens | Ultimate Compressive Strength | | | Elastic Modulus | |
|---|---|---|---|---|---|---|
| | | Arithmetic Mean (psi) | Standard Deviation (psi) | Coef. of Variation (percent) | Mean Static (psi x $10^6$) | Mean Sonic (psi x $10^6$) |
| 3 | 2 | 5,350 | - | - | 3.45 | 4.48 |
| 7 | 11 | 6,680 | 490 | 7.3 | 3.85 | 5.40 |
| 28 | 2 | 7,450 | - | - | 3.80 | 5.64 |

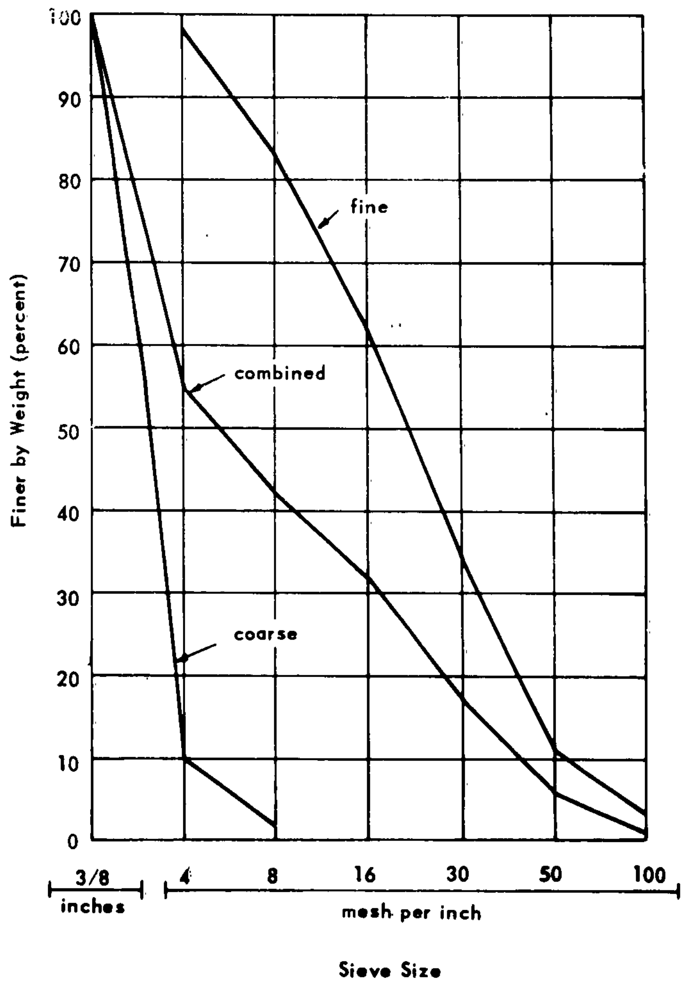



Figure 20. Sieve analysis of aggregates.

Figure 21 shows a plot of the static and sonic moduli for the field-cured cylinders. The sonic readings taken at three years of age were eliminated when it was observed that they had a much larger coefficient of variation (10.1 percent) than any other group, that their average did not lie on what was otherwise a smooth curve, and that the resonant-frequency measuring instrument was found to be giving erroneous readings shortly after this data was obtained.

The results of the tests on the field-cured concrete cylinders were plotted on a semilog scale so as to give the best distribution to the data points. In this manner the large number of tests and the large changes during the first 28 days could be shown on the same curve with the small number of tests and the small changes during the last four years. Also, this method of plotting accentuates any small changes during the latter years.

Figure 21 shows that the static modulus increased rapidly during the first two months and then fairly steadily after that at a rate of about 200,000 psi per year. The sonic modulus increased rapidly during the first week and then fell off slightly during the next three months. It then increased at a rate of about 100,000 psi per year. The two moduli therefore appear to be converging.

Compressive Strength. An exceptionally high ultimate compressive strength was developed in the concrete at an early age. The results of the tests on the fog-cured cylinders are given in Table IV. A plot of the ultimate compressive strength of the field-cured concrete cylinders versus age is shown in Figure 22. Even at three days the field-cured cylinders had a strength of 5,270 psi. At ninety days it was almost 9,000 psi, and after two years it was above 10,000 psi.

Modulus of Rupture. Information on the modulus of rupture of the concrete is presented in Table V. These values for the field-cured beams were less than the normally expected 10 percent of ultimate compressive strength.

Table V. Results from Modulus of Rupture Beams

| Age (days) | Type of Cure | Number of Specimens | Modulus of Rupture | | |
|---|---|---|---|---|---|
| | | | Arithmetic Mean (psi) | Standard Deviation (psi) | Coef. of Variation (percent) |
| 7 | fog | 9 | 740 | 42 | 5.6 |
| 7 | field | 9 | 533 | 38 | 7.1 |
| 28 | field | 8 | 500 | 49 | 9.8 |

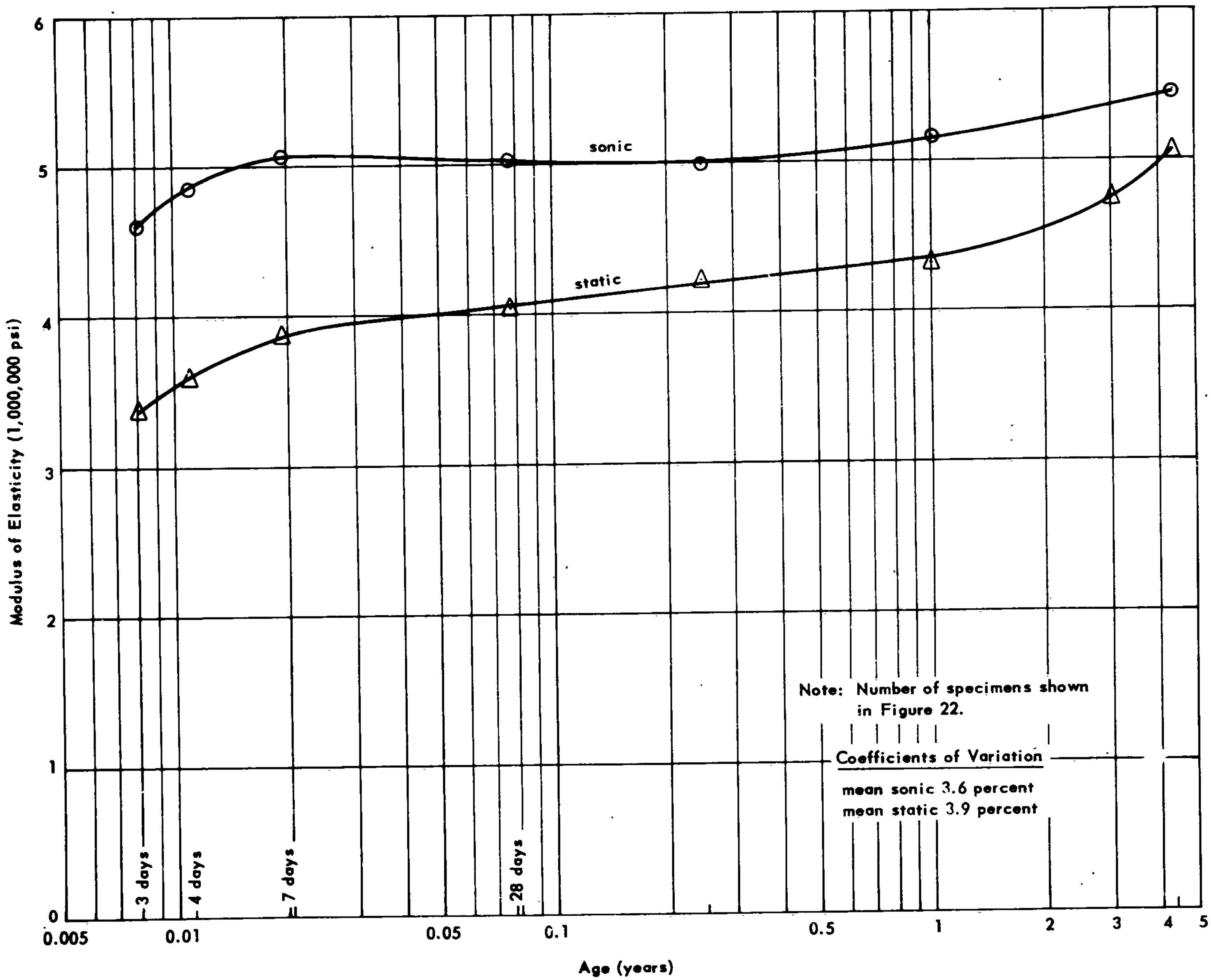



Figure 21. Modulus of elasticity of field-cured concrete cylinders.

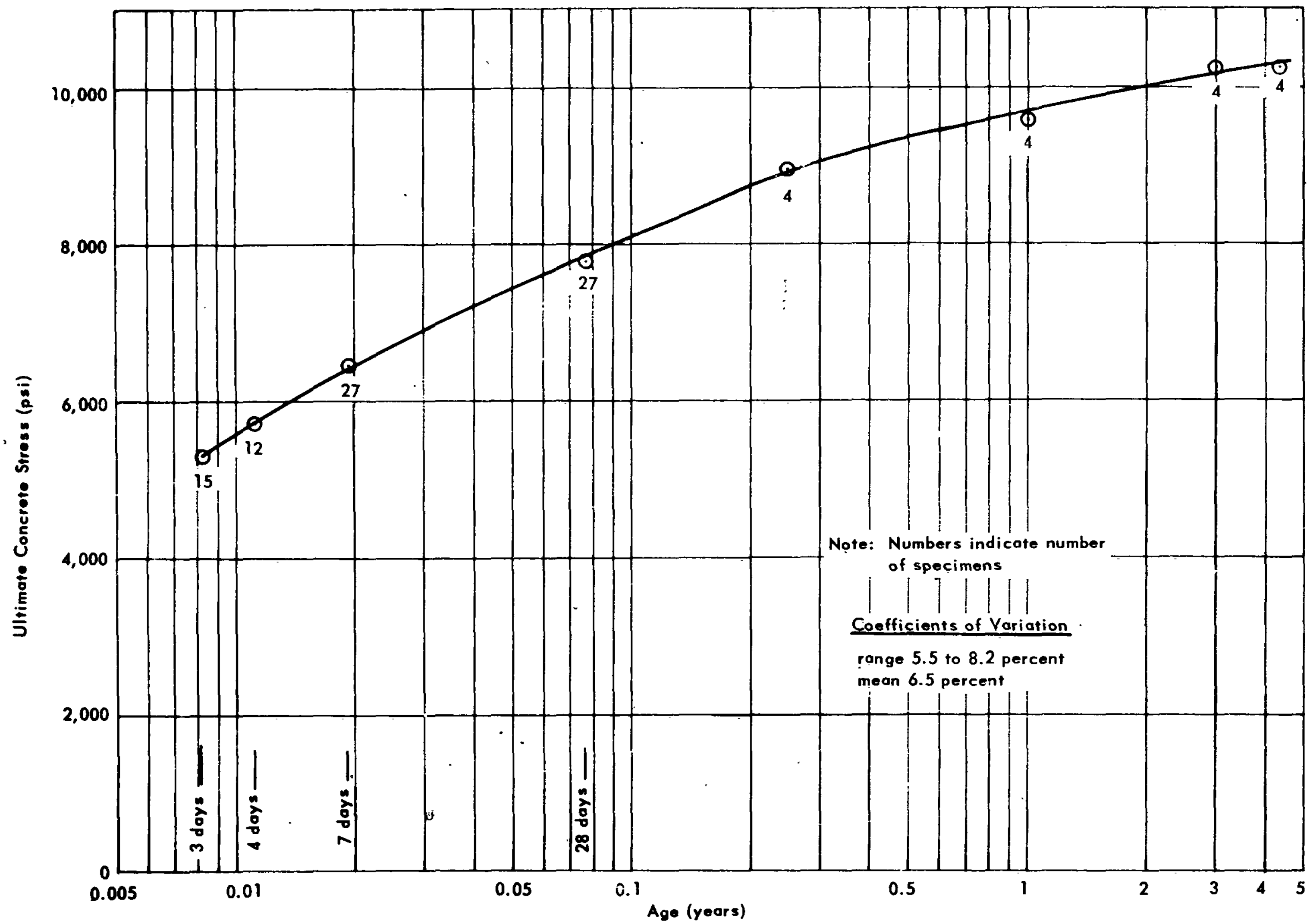



Figure 22. Ultimate strength of field-cured concrete cylinders.

Coefficient of Expansion. The coefficient of expansion measurements of the prisms were taken during several cycles where the temperature was varied between 40 and 72, or 72 and 120 degrees Fahrenheit. Considerable variation in values was obtained both between specimens and between cycles. The average coefficient of expansion was 4.8 microinches per inch per degree Fahrenheit.

Shrinkage. Ultimate shrinkage was determined by taking length measurements of the prisms at 24 hours, again after field-curing for approximately 120 days, and again after oven-drying at 220 degrees Fahrenheit for 6 days. Measurements were also taken after an additional 13 days storage at 50 percent relative humidity. The average shrinkage due to normal drying in the field was 820 microinches per inch and to oven-drying an additional 460 microinches per inch, making an average ultimate shrinkage of 1,280 microinches per inch. These values are very high for concrete. Subsequent to oven-drying, the specimens expanded 230 microinches per inch during 13 days storage at 73.4 degrees Fahrenheit and 50 percent relative humidity.

To obtain values of shrinkage typical of those taking place in the actual beams, two full-sized dummy sections were constructed. They had the same cross section and contained the same amount of mild steel as the beams. They were 5 feet long and were simply supported at the quarter points. To further simulate the conditions in the beams, the ends of the dummy sections were covered with painted plywood to restrict the movement of air and moisture through the otherwise open ends. The sections were instrumented at mid-span with four Carlson strain gages in the same manner as the beams (see Figures 7 and 8). The average shrinkage recorded in these sections is shown in Figure 23.

The cyclic nature of the shrinkage data may be attributed to the normal cyclic changes in the weather. For comparison purposes, the monthly precipitation and average maximum temperature at the nearby Pacific Missile Range Weather Center are also shown in Figure 23.

The monthly average of the relative humidity did not show as good a correlation with the shrinkage data and was not plotted. The average maximum and minimum temperature and relative humidity during the period of testing were as follows:

| | Temperature | | Relative Humidity | |
|---|---|---|---|---|
| | Maximum | Minimum | Maximum | Minimum |
| Winter | 64 F | 47 F | 86% | 45% |
| Summer | 71 F | 58 F | 95% | 65% |

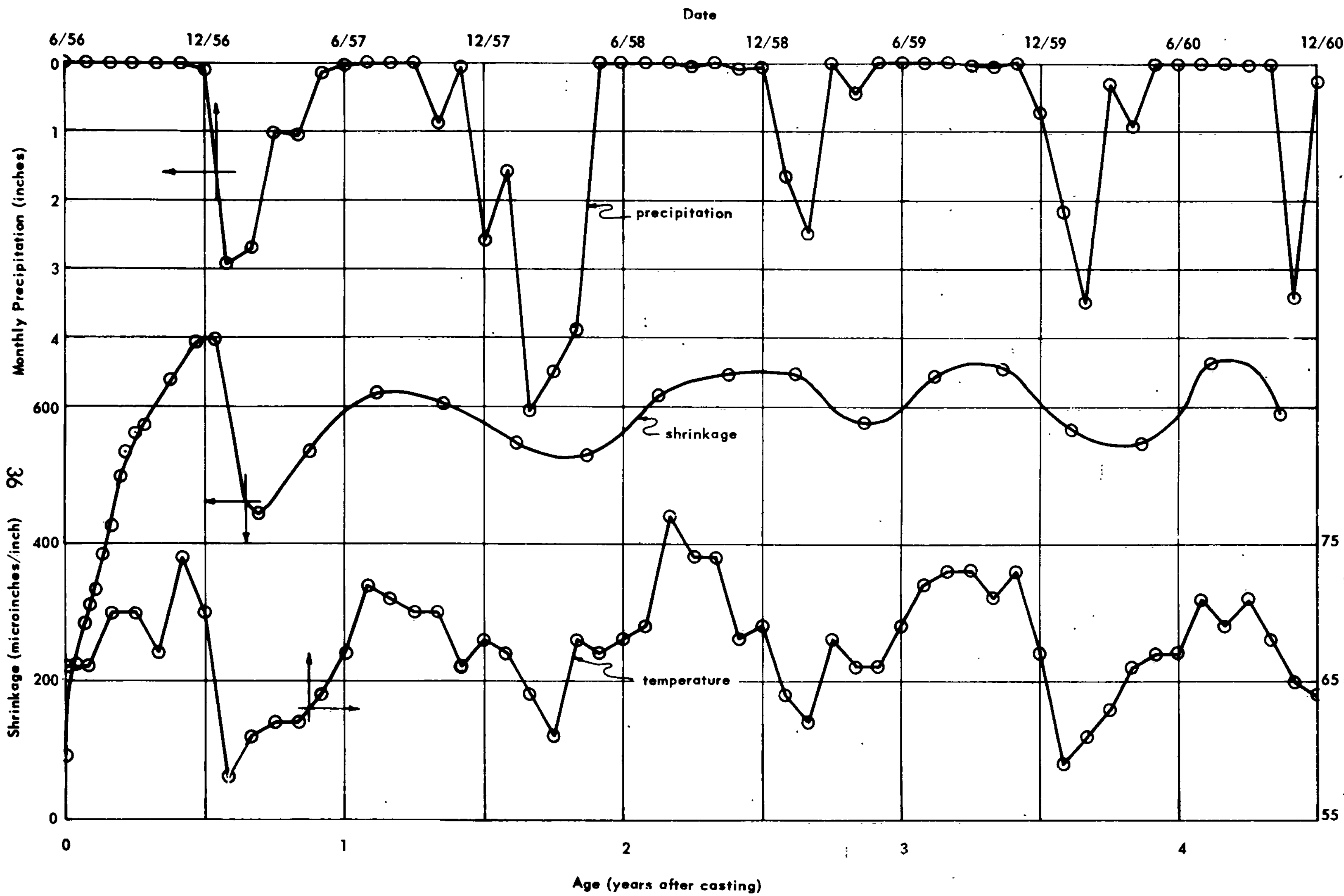



Figure 23. Monthly precipitation, total concrete shrinkage, and average maximum temperature.

## Appendix B

## FABRICATION OF BEAMS

The eight beams were cast in numerical sequence, with one beam completed every 10 or 11 days. Prestressing and loading were carried on concurrently with the casting according to a schedule so that the beams were of the same age when these operations were performed. Thus a beam was prestressed and loaded every 10 or 11 days.

The hollow-box shape of the beams was obtained by welding together two channel sections. These sections were cast on their sides as shown in Figure 24. Both of the channel sections for a given beam were cast in the same forms, with the second section cast on top of and facing the first (bottom) section. This method assured an almost perfect fit between the channels.

Two identical sets of wooden bottom and side forms were used and two sets of inside steel pans. A bottom channel was cast in one set of wooden forms and a top channel in the other. One beam was thereby completed in each casting operation.

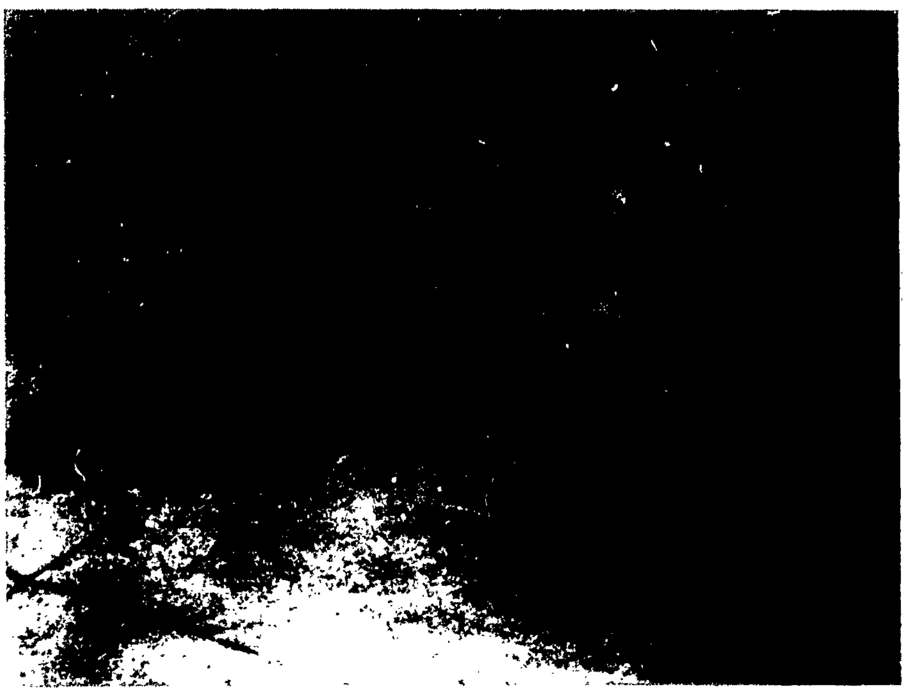


**Figure 24. Wooden forms for beams. Bottom half of beam in right form completed.**

The concrete was mixed in a 16-cubic-foot-capacity Rex rotating-drum mixer. Eight 11-1/2-cubic-foot batches were required per casting operation. The concrete was shoveled into place in the forms and vibrated internally with laboratory-model vibrators. After all exposed surfaces had been finished and had obtained their initial set, they were sprayed with Hunts curing compound and covered over with plywood.

The day following a casting operation the side forms were removed from the completed beam and the bottom pans from the half-finished beam. Curing compound was applied to the newly exposed surfaces.

Three or four days after being cast, the top channel section was lifted by vacuum processes and set on steel horses as shown in Figure 25. The top pans were then removed. The prestressing tendons were installed and thoroughly covered with a special grease to prevent corrosion. Care had to be exercised during this step to insure that all sides of each wire were completely covered. The top channel section was then set back into place with the vacuum-lift equipment and the two channels welded together at the 18 steel inserts provided for this purpose. Figure 26 shows a typical channel connection being welded. Upon completion of the welding, the vacuum equipment was used to move the beam off the forms. The recesses for the channel connections were then sand-blasted, 2 x 2-inch mesh tacked in place, and the recesses filled with a dry packed-concrete mix. One channel was allowed to cure a total of 28 days, and the other 17 or 18 days, before the beam was prestressed.

Prestressing of a beam was started at 0700 so that it could be finished before concrete strain readings were significantly affected by the heat from strong sunlight. The center of the three prestressing units was stressed to one-half of its total stress while the beam was still on its side. The beam was then rotated into its normal vertical position by the method shown in Figure 27. The two other prestressing units were then simultaneously stressed, as shown in Figure 28, to about 3 percent over their design value.

When the end nuts were set and the center unit brought up to its full value, all three units were within 1/2 percent of their initial design prestressing value of 86.7 kips. After the prestressing had been completed, the beam was moved into position ready for loading.

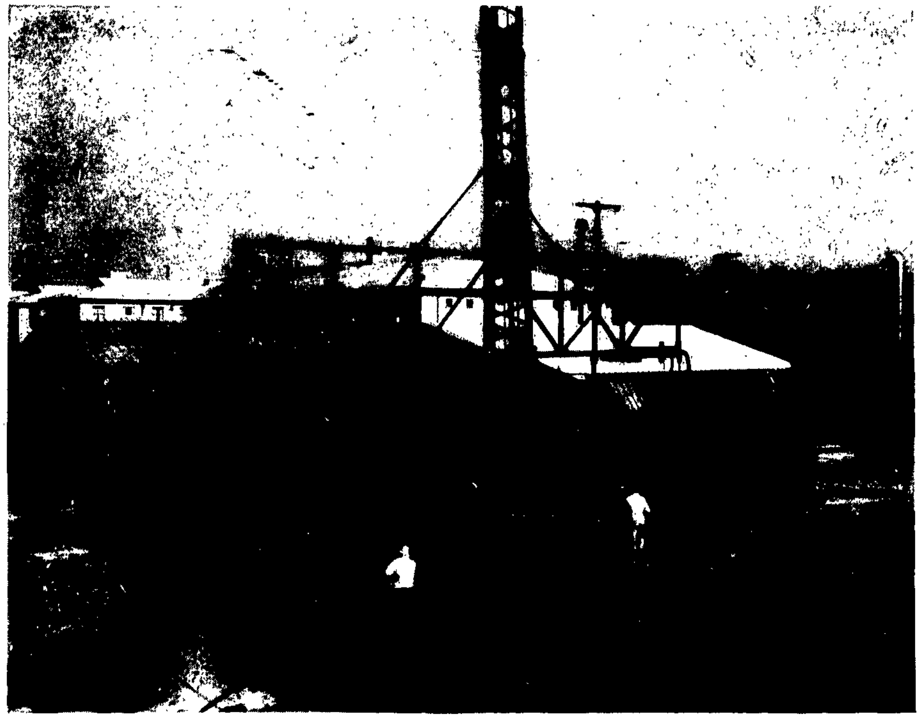


Figure 25. Vacuum equipment used for lifting.

Figure 26. Welding of channel sections.

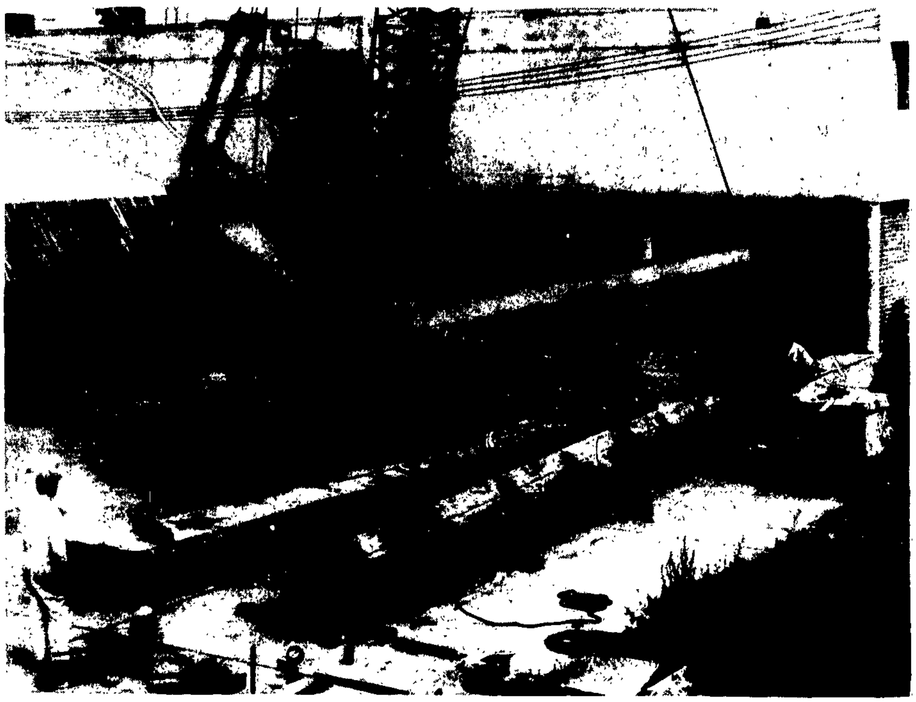


Figure 27. Rotating beam into vertical position.

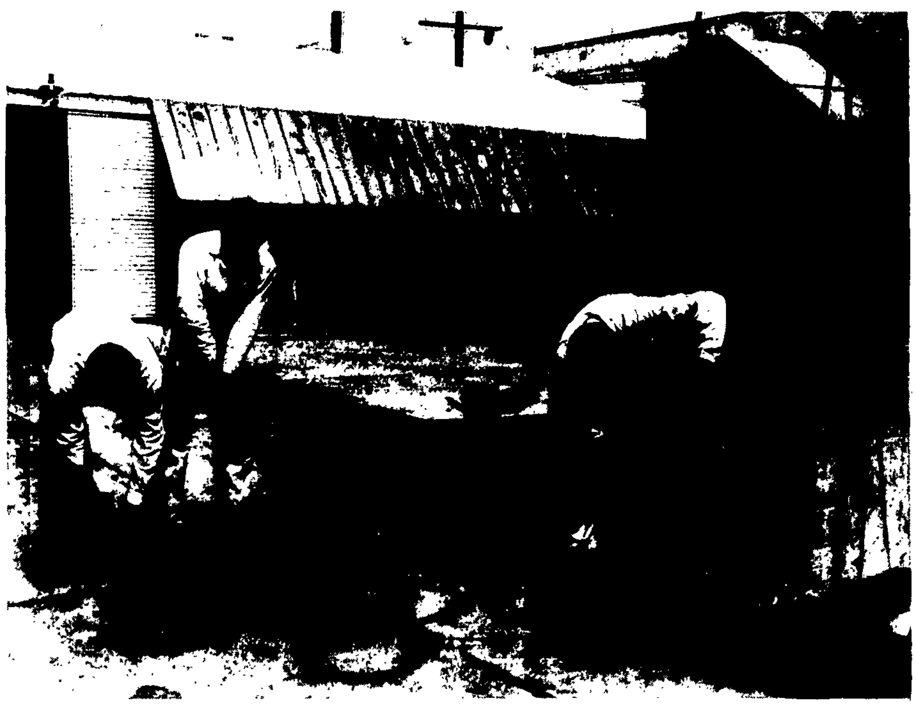


Figure 28. Prestressing.

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